Washington State Department of Transportation

SEG 1B
RETAINING WALL 06.50L

I-405; RENTON TO BELLEVUE WIDENING AND EXPRESS TOLL LANES PROJECT

Design Calculations:

FINAL SUBMITTAL

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PARSONS

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RETAINING WALL 06.50L

DESCRIPTION

RETAINING WALL 06.50L

1.0 – Soil Nail Fascia and CIP Finish Design

[A] BASIS

To design a the Shotcrete and CIP Fascia for soil nail walls according to the FHWA Soil Nail Walls Reference Manual.

This calculation applies to walls: 6.50L

[B] REFERENCES

Acronym	Source
AASHTO	AASHTO LRFD 2017 8th Edition
BDM	WSDOT Bridge Design Manual M23-50.18, June 2018
FHWA	FHWA GEC 7 Soil Nail Walls Reference Manual
GEM	Wall Package 1 - Retaining Wal 6.50L Geotechnical Design Memo, Oct. 2021

[C] DESIGN NOTES AND PARAMETERS

The initial shotcrete fascia is designed for all the loads. Per GEC 7, the CIP fascia must also be designed for all loads. We believe the FHWA design guide is over-conservative in applying the full load to the CIP fascia section, and also in reducing the stud resistance beyond what is typically done, but it is mandated in the RFP. Therefore, this calculation follows the design guide verbatim.

[D] MATERIAL PROPERTIES

γ_c =	150.0 pcf		{concrete unit weight for E_c determination}	[BDM]
γ_{rc} =	155.0 pcf		{reinforced concrete unit weight}	[BDM]
γ_{ss} =	490.0 pcf		{structural steel unit weight}	[BDM]
g =	32.2 lbm-s²/ft⁴		{gravitational acceleration constant}	
m_{rc} =	4.8 lbm-s²/ft⁴	= γ_{cw} / g	{unit mass of reinforced concrete}	

Concrete									
Type	f'_c	K_1	E_c	f'_{ce}	E_{ce}	α_{TU}	υ	G_c	G_{ce}
Text	ksi	#	ksi	ksi	ksi	°F-1	#	ksi	ksi
Reinforced	4.00	1.00	4,266	5.20	4,652	6.00E-06	0.20	1,778	1,938

f'_c =	{concrete compressive strength }	
K_1 =	{correction factor for source of aggregate}	
$E_c = 120000\ K_1(\gamma_c)^2(f'_c)^{0.33}$	{concrete modulus of elasticity}	[AASHTO 5.4.2.4]
$f'_{ce} = 1.3f'_c$	{expected concrete compressive strength}	[SGS 8.4.4-1]
$E_{ce} = 120000\ K_1(\gamma_c)^2(f'_{ce})^{0.33}$	{expected concrete modulus of elasticity}	[AASHTO 5.4.2.4]
α_{TU} =	{coefficient of thermal expansion}	[AASHTO 5.4.2.2]
υ =	{poisson's ratio}	[AASHTO 5.4.2.5]
$G_c = E_c / (2*(1+υ))$	{concrete shear modulus}	
$G_{ce} = E_{ce} / (2*(1+υ))$	{concrete expected shear modulus}	

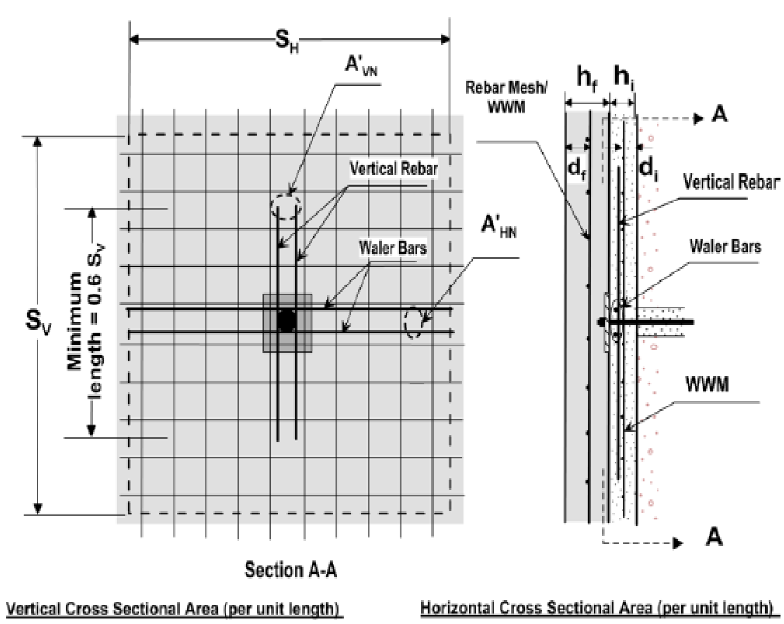
ASTM A706 Grade 60 Reinforcing Steel							
Bar Size	f_y	f_u	E_s	f_{ye}	f_{ue}	ε_y	ε_{ye}
#	ksi	ksi	ksi	ksi	ksi	#	#
All	60	80	29000	68	95	0.0021	0.0023

f_y =		{minimum yield strength}	[ASTM A706-16 Table A1.2]
f_u =		{minimum tensile strength}	[ASTM A706-16 Table A1.2]
E_s =		{steel reinforcement modulus of elasticity}	[AASHTO 5.4.3.2]
f_{ye} =		{expected minimum yield strength}	[SGS Table 8.4.2-1]
f_{ue} =		{expected minimum tensile strength}	[SGS Table 8.4.2-1]
ε_y =	f_y / E_s	{nominal yield strain}	
ε_{ye} =		{expected yield strain}	[SGS Table 8.4.2-1]
ε_{tl} =	0.005	{tension-controlled reinf. steel strain limit}	[AASHTO 5.7.2.1]
ε_{cl} =	0.002	{compression-controlled reinf. steel strain limit}	[AASHTO 5.7.2.1]
ε_c =	0.003	{maximum usable concrete compression strain}	[AASHTO 5.7.2.1]

Welded Wire Reinforcement							
Bar Size	f_y	f_u	E_s	f_{ye}	f_{ue}	ε_y	ε_{ye}
#	ksi	ksi	ksi	ksi	ksi	#	#
<W1.2	56	70	29000	68	95	0.0019	0.0023
>W1.2	65	75	29000	68	95	0.0022	0.0023
D	70	80	29000	68	95	0.0024	0.0023

[E] GEOMETRY

Initial facing thickness (hi) =	6.00 in	{shotcrete}	**Studs:**	
Finish facing thickness (hf) =	8.50 in		Headed studs:	
Length of square bearing plate, L_{bp} =	10.00 in		Actual length =	5.38 in
Thickness of bearing plate =	1.50 in		Head diameter =	1.25 in
Soil nail horizontal spacing S_H=	5.00 ft		Shaft diameter =	0.75 in
Soil nail vertical spacing S_V=	4.00 ft		Head thickness =	0.38 in



[F] FACING BENDING/FLEXURAL RESISTANCE PER FHWA 6.6.5

[FHWA 6.6.5a]

Initial Nominal Flexural Resistance, at (M)id-span Between Nails					
Direction	Mesh	reinf. area	a_{ij}	ρ_{ij}	initial m_{ij}
Text	#	in²/ft	in²/ft	%	kip-ft/ft
Vertical (V)	4x4 - W4.0 x	0.120	a vm=0.120	ρ vm=0.33	1.89
Horizontal (H)	4x4 - W4.0 x	0.120	a hm=0.120	ρ hm=0.33	1.89

a_{ij} = {ratio of cross-sectional area of reinforcement per unit width (in "i" direction and "j" location) and h = thickness of the facing being designed, whether initial or final. The direction "i" can be "v" (for vertical) or "h" (for horizontal); the location "j" can be "n" (nail head) or "m" (mid-span between nails).}

ρ_{ij} = {reinforcement ratio}

m_{ij} = {bending resistance of facing}

[FHWA 6.6.5b]

Initial Nominal Flexural Resistance, at (N)ails						
Direction	Anchor bar size	# Anchor bars	Anchor bar area	a_{ij}	ρ_{ij}	initial m_{ij}
Text	#	#	in²	in²/ft	%	kip-ft/ft
Vertical (V)	5	4	1.14	a vn=0.35	ρ vn=0.97	5.14
Horizontal (H)	5	4	1.14	a hn=0.41	ρ hn=1.13	5.89

φ_{FF} =	0.90
γ_{EV} =	1.35

Check that the max. nail head force (static or seismic) does not exceed the nail head force at max. bending of the facing:

[FHWA 6.6.5b]

Initial Condition Fascia Flexure Check							
Direction	Initial C_F	Initial R_{FF}	$T_{0,stat}$	$T_{0,seis}$	$\varphi_{FF} R_{FF}$	$\gamma_{EV} T_0$	D / C
Text	#	kip	kip	kip	kip	kip	#
Vertical (V)	1.50	105.5	15.0	15.0	94.9	20.3	0.213
Horizontal (H)	1.50	74.7			67.2	20.3	0.301

C_F = {factor that considers the effect of non-uniform soil pressures acting behind the facing}

R_{FF} = {nail head force at maximum bending -nominal bending resistance}

T_0 = {maximum (unfactored) tensile force at soil nail head, for static or seismic load cases}

$\gamma_{EV} T_0$ = γ_{EV}*Max($T_{0,stat}$,$T_{0,seis}$) {total factored load on initial fascia at nail}

Max/Min Reinforcing Check				
ρ_{min}	ρ_{max}	ρ_{tot}	D / C	
%	%	%	#	
0.25	2.00	0.97	0.485	OK

[FHWA 6.6.5b]

Final Nominal Flexural Resistance, at (M)id-span Between Nails						
Direction	Bar size	Bar area	Bar spacing	a_{ij}	ρ_{ij}	final m_{ij}
Text	#	in²	in	in²/ft	%	kip-ft/ft
Vertical (V)	6	0.44	12.00	a vm=0.44	ρ vm=0.86	8.64
Horizontal (H)	6	0.44	12.00	a hm=0.44	ρ hm=0.86	8.64

[FHWA 6.6.5b]

Final Nominal Flexural Resistance, at (N)ail Head						
Direction	Add'l Bar size	Add'l Bar area	Add'l Bar	a_{ij}	ρ_{ij}	final m_{ij}
Text	#	in²	in	in²/ft	%	kip-ft/ft
Vertical (V)	0	0.00	12.00	a vn=0.44	ρ vn=0.86	8.64
Horizontal (H)	0	0.00	12.00	a hn=0.44	ρ hn=0.86	8.64

φ_{FF} = 0.90

<u>Check that the max. nail head force (static or seismic) does not exceed the nail head force at max. bending of the facing:</u>

[FHWA 6.6.5b]

Final Condition Fascia Flexure Check					
Direction	final C_F	final R_{FF}	Alt. R_{FF}*	$\gamma_{EV}T_O$	D / C
Text	#	kip	kip	kip	#
Vertical (V)	1.0	172.8	113.7	20.3	0.198
Horizontal (H)	1.0	110.6	113.7	20.3	0.203

*Alt RFF applies when WWM and rebar are Grade 60 and f'c = 4000psi

Max/Min Reinforcing Check				
ρ_{min}	ρ_{max}	ρ_{tot}	D / C	
%	%	%	#	
0.25	2.00	0.86	0.431	OK

[G] WALL CAP AND FENCE LOADING

Check fence loads on fascia at nail head

P_{fp} =	200.00 lb	{point load in any direction at top of post}	[RDM M22-01 730.04(7)b]
w_{fp} =	50.00 plf	{pedestrian LL distributed load on fabric)	[AASHTO 13.8.2]
P_{FP} =	600.00 lb	{total LL point load at top of post}	

P_d =	15.00 psf	{wind distributed load on fabric}
P_{WS} =	480.00 lb	{total WS point load at top of post}

Combo	LL	WS	
STR V	1.35	1.00	{controls}

P =	1.29 k	{total load at top of post}
d_{nail} =	9.75 ft	{maximum distance from top of post to top-most nail (nail 57)}
M_u =	11.62 k-ft	{moment demand at nail head}
$m_{ij\ (final)}$ =	8.64 k-ft/ft	{vertical bending resistance at nail head}
$h_{trib,min}$ =	5.00 ft	{minimum tributary width of soil nail}
M_n =	43.19 k-ft	{vertical moment capacity at nail head}
D/C =	0.269	

[H] EDGE OF PANEL BENDING MOMENT

Check that the wall panels can resist the outermos Check nails 46 & 47

P =	40.50 k	{total nail force on edge of panel}
b_{trib} =	4.50 ft	{tributary width}
h_{trib} =	7.42 ft	{wall height}
p_{nail} =	1.21 ksf	{pressure behind wall due to nails}

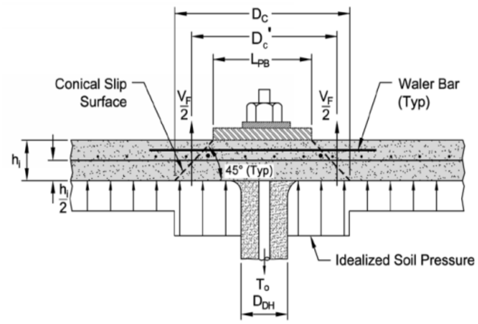
per 1' strip height:

w_{nail} =	1.21 klf
d_{edge} =	3.00 ft
M_{nail} =	5.46 k-ft

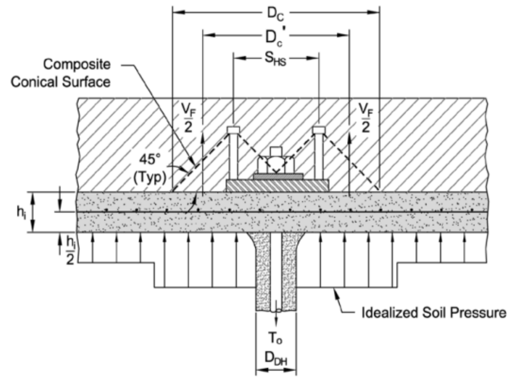
$m_{ij\ (initial)}$ =	5.89 k-ft/ft	{horizontal moment capacity at nail head}
$h_{trib,min}$ =	4.00 ft	{minimum tributary height of soil nail}
M_n =	23.55 k-ft	{horizontal moment capacity at nail head}
D/C =	0.232	

[I] FACING PUNCHING SHEAR RESISTANCE

[FHWA 6.6.6]



(a) Bearing Plate Connection



(b) Headed Stud Connection

φ_{FP} = 0.90

Initial Facing: Bearing Plate Connection						
C_P	D'_c	h_i	V_F	$\varphi_{FP} R_{FP}$	$\gamma_{EV} T_O$	D / C
#	ft	ft	kip	kip	kip	#
1.00	1.33	0.50	76.8	69.1	20.3	0.29

D'_c = {effective equivalent diameter of conical slip surface} = $L_{bp} + h_i$

V_F = {concrete punching shear basic resistance acting through the facing section} = 0.58 SQRT(f'c) π D'c hc

C_P = {dimensionless factor that accounts for the contribution of the soil support under the nail head to the shear resistance}

R_{FP} = {nominal punching shear resistance at facing} = $C_P V_F$

Final Facing: Headed Stud Connection									
S_{SH}	t_{SH}	L_S	h_c	D'_c	V_F	C_P	$\varphi_{FP} R_{FP}$	$\gamma_{EV} T_O$	D / C
in	in	in	in	ft	kip	#	kip	kip	#
7.00	0.375	5.375	6.50	1.08	67.6	1.00	60.9	20.3	0.333

$D'_c = \mathrm{Min}(S_{SH} + h_c, 2h_c)$

S_{SH} = {head stud spacing}

h_c = {effective depth of the conical surface} = $L_s + t_p - t_{SH}$

t_p = {bearing plate thickness}

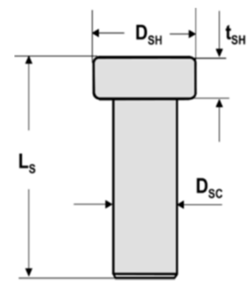
[J] FACING HEADED STUD TENSILE RESISTANCE

φ_{FH} =	0.70
γ_{stat} =	1.35

Initial Condition					
N_H	Stud Head Area, A_S	$f_{y\text{-}hs}$	$\varphi_{FH}\,R_{FH}$	γ_{T0}	D / C
#	in²	ksi	kip	kip	#
6	0.44	60.00	111.3	20.3	0.182

N_H = {number of headed studs}
$f_{y\text{-}hs}$ = {yield strength of headed studs}

Compression on Concrete Behind Stud Head							
$2.5A_S$	A_H	D / C	D_S	D_H	$0.5(D_H\text{-}D_S)$	t_{SH}	D / C
sq.in	in²	#	in	in	in	in	#
1.10	1.23	0.900	1.25	0.75	0.25	0.375	0.667



Geometry of Headed Stud

D_{SH} = {diameter of stud head}
A_H = {cross sectional area of the stud head}

Cover over Studs	Min Cover*	D / C
in	in	#
1.63	1.50	0.923

*Minimum Cover is Per WSDOT Std Dwg 8.1-A4-3, not the 2" required by FHWA

[A] BASIS

To design a the Shotcrete and CIP Fascia for soil nail walls according to the FHWA Soil Nail Walls Reference Manual.

This calculation applies to walls: 6.50L (under roadway section)

[B] REFERENCES

Acronym	Source
AASHTO	AASHTO LRFD 2017 8th Edition
BDM	WSDOT Bridge Design Manual M23-50.18, June 2018
FHWA	FHWA GEC 7 Soil Nail Walls Reference Manual
GEM	Wall Package 1 - Retaining Wal 6.50L Geotechnical Design Memo, Oct. 2021

[C] DESIGN NOTES AND PARAMETERS

The initial shotcrete fascia is designed for all the loads. Per GEC 7, the CIP fascia must also be designed for all loads. We believe the FHWA design guide is over-conservative in applying the full load to the CIP fascia section, and also in reducing the stud resistance beyond what is typically done, but it is mandated in the RFP. Therefore, this calculation follows the design guide verbatim.

[D] MATERIAL PROPERTIES

γ_c =	150.0 pcf	{concrete unit weight for E_c determination}	[BDM]
γ_{rc} =	155.0 pcf	{reinforced concrete unit weight}	[BDM]
γ_{ss} =	490.0 pcf	{structural steel unit weight}	[BDM]

g =	32.2 lbm-s²/ft⁴		{gravitational acceleration constant}
m_{rc} =	4.8 lbm-s²/ft⁴	$= \gamma_{cw} / g$	{unit mass of reinforced concrete}

Concrete									
Type	f'_c	K_1	E_c	f'_{ce}	E_{ce}	α_{TU}	υ	G_c	G_{ce}
Text	ksi	#	ksi	ksi	ksi	°F-1	#	ksi	ksi
Reinforced	4.00	1.00	4,266	5.20	4,652	6.00E-06	0.20	1,778	1,938

f'_c =	{concrete compressive strength }	
K_1 =	{correction factor for source of aggregate}	
$E_c = 120000\ K_1(\gamma_c)^2(f'_c)^{0.33}$	{concrete modulus of elasticity}	[AASHTO 5.4.2.4]
$f'_{ce} = 1.3f'_c$	{expected concrete compressive strength}	[SGS 8.4.4-1]
$E_{ce} = 120000\ K_1(\gamma_c)^2(f'_{ce})^{0.33}$	{expected concrete modulus of elasticity}	[AASHTO 5.4.2.4]
α_{TU} =	{coefficient of thermal expansion}	[AASHTO 5.4.2.2]
υ =	{poisson's ratio}	[AASHTO 5.4.2.5]
$G_c = E_c / (2*(1+\upsilon))$	{concrete shear modulus}	
$G_{ce} = E_{ce} / (2*(1+\upsilon))$	{concrete expected shear modulus}	

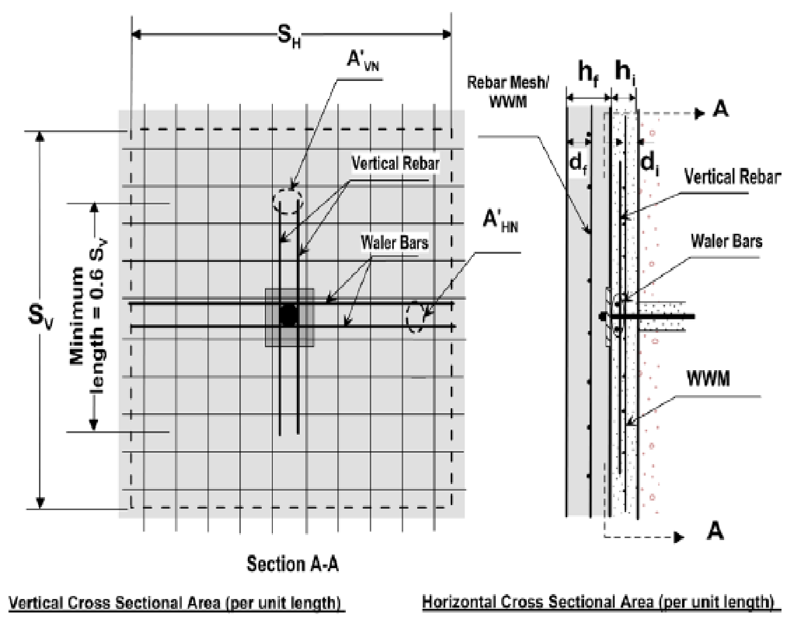
ASTM A706 Grade 60 Reinforcing Steel							
Bar Size	f_y	f_u	E_s	f_{ye}	f_{ue}	ε_y	ε_{ye}
#	ksi	ksi	ksi	ksi	ksi	#	#
All	60	80	29000	68	95	0.0021	0.0023

f_y =		{minimum yield strength}	[ASTM A706-16 Table A1.2]
f_u =		{minimum tensile strength}	[ASTM A706-16 Table A1.2]
E_s =		{steel reinforcement modulus of elasticity}	[AASHTO 5.4.3.2]
f_{ye} =		{expected minimum yield strength}	[SGS Table 8.4.2-1]
f_{ue} =		{expected minimum tensile strength}	[SGS Table 8.4.2-1]
ε_y =	f_y / E_s	{nominal yield strain}	
ε_{ye} =		{expected yield strain}	[SGS Table 8.4.2-1]
ε_{tl} =	0.005	{tension-controlled reinf. steel strain limit}	[AASHTO 5.7.2.1]
ε_{cl} =	0.002	{compression-controlled reinf. steel strain limit}	[AASHTO 5.7.2.1]
ε_c =	0.003	{maximum usable concrete compression strain}	[AASHTO 5.7.2.1]

Welded Wire Reinforcement							
Bar Size	f_y	f_u	E_s	f_{ye}	f_{ue}	ε_y	ε_{ye}
#	ksi	ksi	ksi	ksi	ksi	#	#
<W1.2	56	70	29000	68	95	0.0019	0.0023
>W1.2	65	75	29000	68	95	0.0022	0.0023
D	70	80	29000	68	95	0.0024	0.0023

[E] GEOMETRY

Initial facing thickness (hi) =	6.00 in	{shotcrete}	**Studs:**	
Finish facing thickness (hf) =	8.50 in		Headed studs:	
Length of square bearing plate, L_{bp} =	10.00 in		Actual length =	5.38 in
Thickness of bearing plate =	1.50 in		Head diameter =	1.25 in
Soil nail horizontal spacing S_H=	4.00 ft		Shaft diameter =	0.75 in
Soil nail vertical spacing S_V=	3.50 ft		Head thickness =	0.38 in



[F] FACING BENDING/FLEXURAL RESISTANCE PER FHWA 6.6.5

[FHWA 6.6.5a]

Initial Nominal Flexural Resistance, at (M)id-span Between Nails					
Direction	Mesh	reinf. area	a_{ij}	ρ_{ij}	initial m_{ij}
Text	#	in²/ft	in²/ft	%	kip-ft/ft
Vertical (V)	4x4 - W4.0 x W4.0	0.120	a vm=0.120	ρ vm=0.33	1.89
Horizontal (H)	4x4 - W4.0 x W4.0	0.120	a hm=0.120	ρ hm=0.33	1.89

a_{ij} = {ratio of cross-sectional area of reinforcement per unit width (in "i" direction and "j" location) and h = thickness of the facing being designed, whether initial or final. The direction "i" can be "v" (for vertical) or "h" (for horizontal); the location "j" can be "n" (nail head) or "m" (mid-span between nails).}

ρ_{ij} = {reinforcement ratio}

m_{ij} = {bending resistance of facing}

[FHWA 6.6.5b]

Initial Nominal Flexural Resistance, at (N)ails						
Direction	Anchor bar size	# Anchor bars	Anchor bar area	a_{ij}	ρ_{ij}	initial m_{ij}
Text	#	#	in²	in²/ft	%	kip-ft/ft
Vertical (V)	5	4	1.14	a vn=0.41	ρ vn=1.13	5.89
Horizontal (H)	5	4	1.14	a hn=0.45	ρ hn=1.24	6.40

φ_{FF} =	0.90
γ_{EV} =	1.35

Check that the max. nail head force (static or seismic) does not exceed the nail head force at max. bending of the facing:

[FHWA 6.6.5b]

Initial Condition Fascia Flexure Check							
Direction	Initial C_F	Initial R_{FF}	$T_{0,stat}$	$T_{0,seis}$	$\varphi_{FF} R_{FF}$	$\gamma_{EV} T_0$	D / C
Text	#	kip	kip	kip	kip	kip	#
Vertical (V)	1.50	106.6	44.0	57.0	96.0	59.4	0.619
Horizontal (H)	1.50	87.0			78.3	59.4	0.758

C_F = {factor that considers the effect of non-uniform soil pressures acting behind the facing}

R_{FF} = {nail head force at maximum bending -nominal bending resistance}

T_0 = {maximum (unfactored) tensile force at soil nail head, for static or seismic load cases}

$\gamma_{EV} T_0$ = γ_{EV}*Max($T_{0,stat}$,$T_{0,seis}$) {total factored load on initial fascia at nail}

Max/Min Reinforcing Check				
ρ_{min}	ρ_{max}	ρ_{tot}	D / C	
%	%	%	#	
0.25	2.00	1.13	0.564	OK

[FHWA 6.6.5b]

Final Nominal Flexural Resistance, at (M)id-span Between Nails						
Direction	Bar size	Bar area	Bar spacing	a_{ij}	ρ_{ij}	final m_{ij}
Text	#	in²	in	in²/ft	%	kip-ft/ft
Vertical (V)	6	0.44	12.00	a vm=0.44	ρ vm=0.86	8.64
Horizontal (H)	6	0.44	12.00	a hm=0.44	ρ hm=0.86	8.64

[FHWA 6.6.5b]

Final Nominal Flexural Resistance, at (N)ail Head						
Direction	Add'l Bar size	Add'l Bar area	Add'l Bar	a_{ij}	ρ_{ij}	final m_{ij}
Text	#	in²	in	in²/ft	%	kip-ft/ft
Vertical (V)	0	0.00	12.00	a vn=0.44	ρ vn=0.86	8.64
Horizontal (H)	0	0.00	12.00	a hn=0.44	ρ hn=0.86	8.64

φ_{FF} = 0.90

<u>Check that the max. nail head force (static or seismic) does not exceed the nail head force at max. bending of the facing:</u>

[FHWA 6.6.5b]

Final Condition Fascia Flexure Check					
Direction	final C_F	final R_{FF}	Alt. R_{FF}*	$\gamma_{EV}T_0$	D / C
Text	#	kip	kip	kip	#
Vertical (V)	1.0	158.0	124.4	59.4	0.531
Horizontal (H)	1.0	120.9	124.4	59.4	0.546

*Alt RFF applies when WWM and rebar are Grade 60 and f'c = 4000psi

Max/Min Reinforcing Check				
ρ_{min}	ρ_{max}	ρ_{tot}	D / C	
%	%	%	#	
0.25	2.00	0.86	0.431	OK

[G] WALL CAP AND FENCE LOADING

Check fence loads on fascia at nail head

P_{fp} =	200.00 lb	{point load in any direction at top of post}	[RDM M22-01 730.04(7)b]
w_{fp} =	50.00 plf	{pedestrian LL distributed load on fabric)	[AASHTO 13.8.2]
P_{FP} =	600.00 lb	{total LL point load at top of post}	

P_d =	15.00 psf	{wind distributed load on fabric}
P_{WS} =	480.00 lb	{total WS point load at top of post}

Combo	LL	WS	
STR V	1.35	1.00	{controls}

P =	1.29 k	{total load at top of post}
d_{nail} =	9.75 ft	{maximum distance from top of post to top-most nail (nail 57)}
M_u =	11.62 k-ft	{moment demand at nail head}
$m_{ij\ (final)}$ =	8.64 k-ft/ft	{vertical bending resistance at nail head}
$h_{trib,min}$ =	4.00 ft	{minimum tributary width of soil nail}
M_n =	34.55 k-ft	{vertical moment capacity at nail head}
D/C =	0.336	

[H] EDGE OF PANEL BENDING MOMENT

Check that the wall panels can resist the outermost nai Check nails 46 & 47

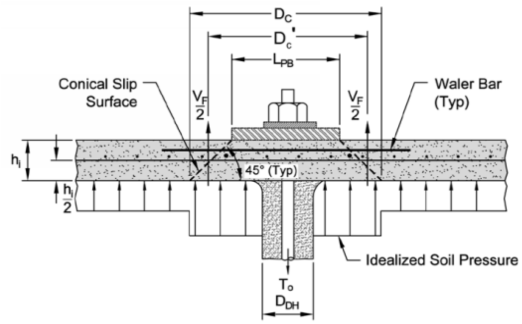
P =	118.80 k	{total nail force on edge of panel}
b_{trib} =	4.50 ft	{tributary width}
h_{trib} =	7.42 ft	{wall height}
p_{nail} =	3.56 ksf	{pressure behind wall due to nails}

per 1' strip height:

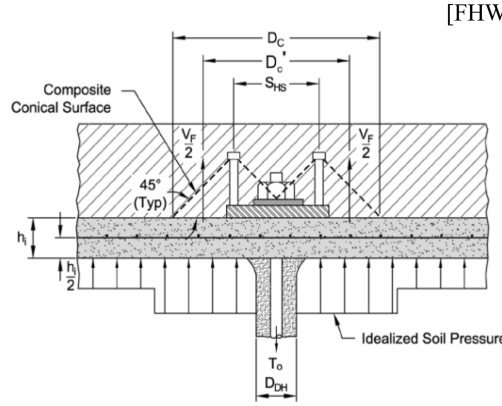
w_{nail} =	3.56 klf
d_{edge} =	3.00 ft
M_{nail} =	16.02 k-ft

$m_{ij\ (initial)}$ =	6.40 k-ft/ft	{horizontal moment capacity at nail head}
$h_{trib,min}$ =	3.50 ft	{minimum tributary height of soil nail}
M_n =	22.41 k-ft	{horizontal moment capacity at nail head}
D/C =	0.715	

[I] FACING PUNCHING SHEAR RESISTANCE [FHWA 6.6.6]



(a) Bearing Plate Connection



(b) Headed Stud Connection

φ_{FP} = 0.90

Initial Facing: Bearing Plate Connection						
C_P	D'_c	h_i	V_F	$\varphi_{FP} R_{FP}$	$\gamma_{EV} T_0$	D / C
#	ft	ft	kip	kip	kip	#
1.00	1.33	0.50	76.8	69.1	59.4	0.86

D'_c = {effective equivalent diameter of conical slip surface} = $L_{bp} + h_i$

V_F = {concrete punching shear basic resistance acting through the facing section} = 0.58 SQRT(f'c) π D'c hc

C_P = {dimensionless factor that accounts for the contribution of the soil support under the nail head to the shear resistance}

R_{FP} = {nominal punching shear resistance at facing} = $C_P V_F$

Final Facing: Headed Stud Connection									
S_{SH}	t_{SH}	L_S	h_c	D'_c	V_F	C_P	$\varphi_{FP} R_{FP}$	$\gamma_{EV} T_0$	D / C
in	in	in	in	ft	kip	#	kip	kip	#
7.00	0.375	5.375	6.50	1.08	67.6	1.00	60.9	59.4	0.976

$D'_c = \text{Min}(S_{SH} + h_c, 2h_c)$

S_{SH} = {head stud spacing}

h_c = {effective depth of the conical surface} = $L_s + t_p - t_{SH}$

t_p = {bearing plate thickness}

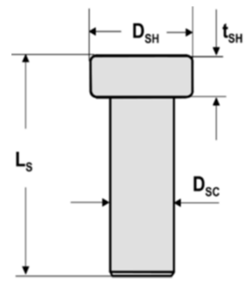
[J] FACING HEADED STUD TENSILE RESISTANCE

φ_{FH} =	0.70
γ_{stat} =	1.35

Initial Condition					
N_H	Stud Head Area, A_S	$f_{y\text{-}hs}$	$\varphi_{FH}\ R_{FH}$	γ_{TO}	D / C
#	in²	ksi	kip	kip	#
6	0.44	60.00	111.3	77.0	0.691

N_H = {number of headed studs}
$f_{y\text{-}hs}$ = {yield strength of headed studs}

Compression on Concrete Behind Stud Head							
$2.5A_S$	A_H	D / C	D_S	D_H	$0.5(D_H\text{-}D_S)$	t_{SH}	D / C
sq.in	in²	#	in	in	in	in	#
1.10	1.23	0.900	1.25	0.75	0.25	0.375	0.667



Geometry of Headed Stud

D_{SH} = {diameter of stud head}
A_H = {cross sectional area of the stud head}

Cover over Studs	Min Cover*	D / C
in	in	#
1.63	1.50	0.923

*Minimum Cover is Per WSDOT Std Dwg 8.1-A4-3, not the 2" required by FHWA

RETAINING WALL 06.50L

2.0 – Retaining Barrier Design

[A] BASIS

- To evaluate the load demands on the retaining barrier and design adequate barrier reinforcement.

[B] REFERENCES

Acronym	Source
AASHTO	AASHTO LRFD Bridge Design Specifications 8th Edition
BDM	WSDOT Bridge Design Manual (M23-50.19) - July 2019
GEM	Wall Package 1 - Retaining Wall 6.50L Geotechnical Design Memo, Oct 2021

[C] DESIGN NOTES AND PARAMETERS

1. The barrier will be evaluated during Strength I, Service I, Extreme I, and Extreme II load combinations.

2. The barrier must be adaquately stable during the Strength I, Service I, and Extreme I load combinations. Extreme II will be ignored for stability calculations because a vehicle impact will be supported by the barrier in either direction of the impact and by engineering judgement, the barrier will not overturn or slide into the adjacent hill.

[D] MATERIAL PROPERTIES

γ_c =	145.0 pcf	{plain concrete unit weight for loads and models}	[BDM Table 3.8.1]
γ_c =	150.0 pcf	{reinforced concrete unit weight for modulus of elasticity}	[BDM 5.1.1D]
γ_{rc} =	155.0 pcf	{reinforced concrete unit weight for loads and models}	[BDM Table 3.8.1]

Plain Concrete									
Elements	f'_c	K_1	E_c	f'_{ce}	E_{ce}	α_{TU}	υ	G_c	G_{ce}
Text	ksi	#	ksi	ksi	ksi	$°F^{-1}$	#	ksi	ksi
Retaining Barrier	4.00	1.00	4266	5.20	4967	6.00E-06	0.20	1778	2070

f'_c = {concrete compressive strength }		
K_1 = {correction factor for source of aggregate}		
$E_{rc} = 120000\ K_1(\gamma_{rc})^2(f'_c)^{0.33}$	{concrete modulus of elasticity}	[AASHTO 5.4.2.4]
$f'_{c,e} = 1.3f'_c$	{expected concrete compressive strength}	[SGS 8.4.4-1]
$E_{rc,e} = 120000\ K_1(\gamma_{rc})^2(f'_{c,e})^{0.33}$	{expected concrete modulus of elasticity}	[AASHTO 5.4.2.4]
α_{TU} = {coefficient of thermal expansion}		[DCM 8.4.2.I.3]
υ = {poisson's ratio}		[AASHTO 5.4.2.5]
$G_c = E_{rc} / (2*(1+\upsilon))$	{concrete shear modulus}	
$G_{ce} = E_{rc,e} / (2*(1+\upsilon))$	{concrete expected shear modulus}	

ASTM A706 Grade 60 Reinforcing Steel							
Bar Size	f_y	f_u	E_s	f_{ye}	f_{ue}	ε_y	ε_{ye}
#	ksi	ksi	ksi	ksi	ksi	#	#
All	60	80	29000	68	95	0.0021	0.0023

f_y = {minimum yield strength} [ASTM A706-16 Table A1.2]

f_u = {minimum tensile strength} [ASTM A706-16 Table A1.2]

E_s = {steel reinforcement modulus of elasticity} [AASHTO 5.4.3.2]

f_{ye} = {expected minimum yield strength} [SGS Table 8.4.2-1]

f_{ue} = {expected minimum tensile strength} [SGS Table 8.4.2-1]

$\varepsilon_y = f_y / E_s$ {nominal yield strain}

ε_{ye} = {expected yield strain} [SGS Table 8.4.2-1]

ε_{tl} = 0.005 {tension-controlled reinf. steel strain limit} [AASHTO 5.6.2.1]

ε_{cl} = 0.002 {compression-controlled reinf. steel strain limit} [AASHTO 5.6.2.1]

ε_c = 0.003 {maximum usable concrete compression strain} [AASHTO 5.6.2.1]

Soil						
Soil Type	γ_s	ϕ_{soil}	$\tan\delta = \tan\phi$	K_a	K_{AE}	K_p
#	pcf	deg	-	#	#	#
Fill	125.0	36	0.73	0.35	0.79	6.00

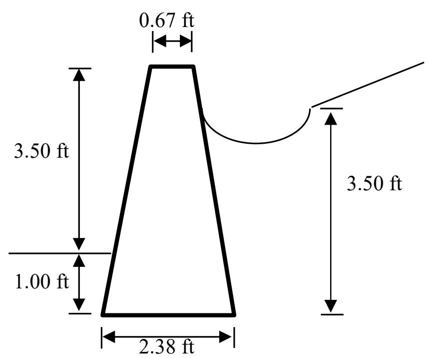
γ_s = {soil unit weight}

ϕ_{soil} = {internal friction angle of soil}

$\tan\delta$ = {Coefficient of friction between soil and bottom of footing} = $\tan\varphi$ for cast in place concrete against so [BDM 7.7.4 C]

[E] GEOMETRY

Height above Roadway	Embedment Depth	Length	Top Thickness	Bottom Thickness	Soil Depth Behind Barrier	Wall Slope
ft	ft	ft	ft	ft	ft	Run/Rise
3.50	1.00	1.00	0.67	2.38	3.50	0.19



[F] GLOBAL STABILITY

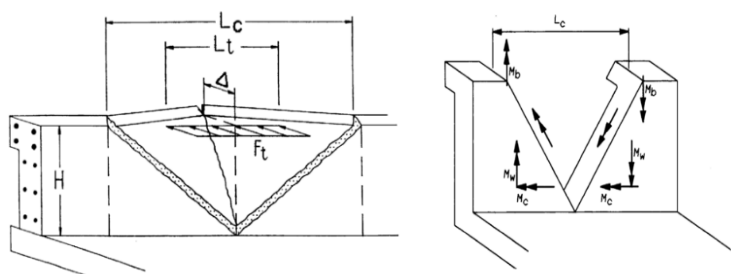
	D / C
CHECK BEARING STRESS:	0.161
CHECK SLIDING:	0.744
CHECK OVERTURNING:	0.980

[G] LOADING

Barrier DC								
Element	Length	Height	Top Thickness	Bottom Thickness	Volume	Weight	Eccentricity CL Ftg	Moment CL Ftg
Text	ft	ft	ft	ft	ft^3 / ft	kip / ft	ft	k-ft / ft
Barrier	1.0	4.50	0.67	2.38	6.8	1.06	0.00	0.00

Lateral Soil Forces EH								
Load	L	H	p_{bot}	p_{top}	F_{long}	y_{bot}	M_{bot}	
Text	ft	ft	ksf	ksf	kip / ft	ft	k-ft / ft	
EH_a	1.00	3.5	0.15	0.00	0.27	1.17	0.31	
EH_p	1.00	1.0	0.75	0.00	-0.38	0.33	-0.13	
$EH_{p, CT, MID}$	1.00	1.5	1.13	0.00	0.84	0.50	0.42	- Ignore top 2ft of soil
$EH_{p, CT, END}$	1.00	0.0	0.01	0.00	0.00	0.00	0.00	- Assume end of barrier has minimal soil behind
EH_{AE}	1.00	3.5	0.35	0.00	0.60	1.17	0.71	

- Lateral vehicle impact loading is applied along length Lt but the portion of barrier that contributes to resisting the load is the length L_c. Length L_c is determined per AASHTO A13.3.1 using the yield line failure pattern and the vertical/longitudinal moment capacities of the barrier.



Lateral Vehicle Impact Forces											
Load	Lateral Test Level	F_t	L_t	Location	Assumed L_c	M_w	M_c	L_c	H_e	F_{long}	M_{bot}
Text	Text	k	ft	Text	ft	k-ft	k-ft	ft	in	k / ft	k-ft / ft
CT	TL-4	54.00	3.50	Middle	6.83	169.29	268.06	6.83	32.00	-7.91	-21.09
				Ends	4.09	169.29	318.15	4.09		-13.22	-35.24

F_t = {transverse vehicle impact load}

L_t = {longitudinal length of distribution of F_t}

M_w = {moment capacity of the longitudinal reinforcement within the barrier}

M_c = {moment capacity of the vertical reinforcement within the barrier accounting for the assumed L_c}

L_c = {critical length or yield line failure pattern}

H_e = {height above roadway surface where the impact faroce is applied}

Lateral Inertial Forces						
Load	A_s	k_h	W	F_{long}	y_{bot}	M_{bot}
Text	g	g	k / ft	k / ft	ft	k-ft / ft
IR_{DC}	0.50	0.25	1.06	0.27	1.83	0.48

[H] LOAD FACTORS

Load Factors						
Limit State	DC max	DC min	EH max	EH min	EQ	CT
Strength I	1.25	0.90	1.50 Active	0.90 Active	0.00	0.00
			1.00 Passive	1.00 Passive		
Service I	1.00	-	1.00	-	0.00	0.00
Extreme Event I	1.00	-	1.00	-	1.00	0.00
Extreme Event II	1.00	-	1.00	-	-	1.00

[I] BARRIER EXTERNAL STABILITY

1. Check footing eccentricity at service and strength limit states [AASHTO 10.6.3.4].

- Eccentricity limits are only applicable at the Strength Limit State per [AASHTO 11.6.2 & 11.6.3], however Service overturning moments and vertical force effects are still summarized below.

- Per [AASHTO 11.6.3.5], passive resistance shal be neglected in stability computations, unless the base of the wall extends below the depth of maximum scour, freeze-thaw, or other disturbances. Where passive resistance is utilized to ensure adequate wall stability, the calculated passive resistance of soil in front of conventional walls shall be sufficient to prevent unacceptable forward movement of the wall. Passive resistance will be considered for extreme events since the roadway will ensure that the barrier won't move on the roadway side and the soil will ensure the barrier won't move on the back face.

- Per [AASHTO 11.6.5.1], 50% of the wall inertial force will be combined with 100% of the seismic active earth loading and 100% of the wall inertial force will be combined with 50% of the seismic active earth loading but no less than the static active earth pressure force.

- Extreme II vehicle impact will be ignored for stability calculations because based on engineering judgement, the section of barrier that is hit will no detach from the rest of the barrier.

Effect	P	M	SERVICE	STRENGTH		EXTREME		
			SER 01	STR 01-01	STR 01-02	EE 01-01	EE 01-02	EE 01-03
			γ	γ Max	γ Min	γ	γ	γ
#	k / ft	k-ft / ft	#	#	#	#	#	#
DC	1.06	0	1.00	1.25	0.90	1.00	1.00	1.00
EH_a	0	0.31	1.00	1.50	0.90	0.00	0.00	1.00
EH_p	0	-0.13	0.00	0.00	0.00	1.00	1.00	1.00
EH_{AE}	0	0.71	0.00	0.00	0.00	1.00	0.50	0.00
IR_{DC}	0	0.48	0.00	0.00	0.00	0.50	1.00	1.00
e	-	-	**3.54 in**	**4.24 in**	**3.54 in**	**9.31 in**	**8.06 in**	**7.61 in**

e_{range}	$e_{min,lim}$	$e_{max,lim}$	e_{max}	Check
in	in	in	in	Text
19.00	-9.500	9.500	4.24	OK

$e_{range} = 2/3B$ {two-thirds eccentricity limit} [AASHTO 11.6.3.3]
$e_{min,lim} = -e_{range} / 2$ {minimum eccentricity limit from center of footing} [AASHTO 11.6.3.3]
$e_{max,lim} = e_{range} / 2$ {maximum eccentricity limit from center of footinge} [AASHTO 11.6.3.3]

- Since live load effects are factored by 0.50 in the EE01 load combination, the eccentricity limit is determined based on linear interpolation between 2/3 and 8/10 of the base of the wall per [AASHTO 11.6.5.1].

$e_{lim,2/3}$	$e_{lim,8/10}$	$\gamma_{LL,EE01}$	e_{range}	$e_{min,lim}$	$e_{max,lim}$	e_{max}	Check
in	in	#	in	in	in	in	Text
19.00	22.80	0.00	19.00	-9.50	9.50	9.31 in	OK

$e_{min} = B_{bot} / 2 - e_{lim} / 2$ {minimum eccentricity limit from center of wall base} [AASHTO 11.6.3.3 & 11.6.5]
$e_{max} = B_{bot} / 2 + e_{lim} / 2$ {maximum eccentricity limit from center of wall base} [AASHTO 11.6.3.3 & 11.6.5]

2. Check footing sliding (cohesionless soil) at service and strength limit states [AASHTO 10.6.3.4].

θ_f =	36.0 °	{internal soil friction angle}	
$\varphi_{\tau,ser}$ =	1.00	{service limit state sliding resistance factor}	[AASHTO 10.5.5.1]
$\varphi_{\tau,str}$ =	0.80	{strength limit state sliding resistance factor}	[AASHTO Table 10.5.5.2.2-1]
$\varphi_{ep,ser}$ =	1.00	{service limit state passive resistance factor}	[AASHTO 10.5.5.1]
$\varphi_{ep,str}$ =	0.50	{strength limit state passive resistance factor}	[AASHTO Table 10.5.5.2.2-1]
$\varphi_{\tau,ee}$ =	1.00	{extreme event limit state sliding friction resistance factor}	[AASHTO 10.5.5.3.3]
$\varphi_{ep,ee}$ =	1.00	{extreme event limit state sliding passive pressure resistance factor}	[AASHTO 10.5.5.3.3]

- Vertical force effects for determining sliding resistance are not factored since a resistance factor is applied to the overall sliding resistance. Vertica live loads are conservatively ignored.

Effect	P	γ
#	k / ft	#
DC	1.06	1.00
EH_a	0	1.00
EH_p	0	1.00
EH_{AE}	0	0.00
IR_{DC}	0	0.00
Rτ	**-**	**0.62 k/ft**

Limit State	R_τ	R_{ep}	**φR**
Text	k / ft	k / ft	k / ft
SER	0.62	0.38	**0.99**
STR	0.62	0.38	**0.68**
EXT	0.62	0.38	**0.99**

φR = {factored sliding resistance}

$R_\tau = |V\tan\theta_f|$ {total wall sliding friction resistance} [AASHTO 10.6.3.4-2]

Effect	F_{long}	SERVICE	STRENGTH		EXTREME		
		SER 01	STR 01-01	STR 01-02	EE 01-01	EE 01-02	EE 01-03
		γ	γ Max	γ Min	γ	γ	γ
#	k / ft	#	#	#	#	#	#
DC	0	1.00	1.25	0.90	1.00	1.00	1.00
EH_a	0.27	1.00	1.50	0.90	0.00	0.00	1.00
EH_p	0	0.00	0.00	0.00	1.00	1.00	1.00
EH_{AE}	0.60	0.00	0.00	0.00	1.00	0.50	0.00
IR_{DC}	0.27	0.00	0.00	0.00	0.50	1.00	1.00
γF	**-**	**0.27 k/ft**	**0.40 k/ft**	**0.24 k/ft**	**0.74 k/ft**	**0.57 k/ft**	**0.53 k/ft**

F_{long} = {horizontal longitudinal applied force effects}

γ = {load factor} [AASHTO Table 3.4.1-1, WSDOT BDM 3.5 & 4.2.6]

Limit State	φR	γF	D/C
Text	k / ft	k / ft	#
SER	**0.99**	**0.27**	**0.27**
STR	**0.68**	**0.40**	**0.59**
EXT	**0.99**	**0.74**	**0.74**

φR = {factored sliding resistance}

γF = {factored sliding demand}

D/C = φF / φR {demand-to-capacity ratio}

[J] CHECK BEARING STRESS

- Bearing resistance is not explicitly required to be checked at the Service limit state per [AASHTO 11.6.2], however it will be checked based on the settlement criteria of 1 in per [GEM].

Limit State	Bearing Capacity	B_{ftg}	q_r	q_u	D/C
Text	#	ft	ksf	ksf	#
SER	11.00 KSF	2.38	11.00	0.59	**0.05**
STR	4.95 KSF		4.95	0.80	**0.16**
EXT	13.00 KSF		13.00	1.29	**0.10**

1. Determine footing bearing pressure demands at Service/Strength/Extreme limit state [AASHTO 11.6.3].

Effect	P	SERVICE	STRENGTH		EXTREME		
		SER 01	STR 01-01	STR 01-02	EE 01-01	EE 01-02	EE 01-03
		γ	γ Max	γ Min	γ	γ	γ
#	k / ft	#	#	#	#	#	#
DC	1.06	1.00	1.25	0.90	1.00	1.00	1.00
EH_a	0	1.00	1.50	0.90	0.00	0.00	1.00
EH_p	0	0.00	0.00	0.00	1.00	1.00	1.00
EH_{AE}	0	0.00	0.00	0.00	1.00	0.50	0.00
IR_{DC}	0	0.00	0.00	0.00	0.50	1.00	1.00
γP	-	1.06 k/ft	1.33 k/ft	0.95 k/ft	1.06 k/ft	1.06 k/ft	1.06 k/ft
e	-	3.54 in	4.24 in	3.54 in	9.31 in	8.06 in	7.61 in
q	-	**0.59 ksf**	**0.80 ksf**	**0.53 ksf**	**1.29 ksf**	**1.03 ksf**	**0.96 ksf**

$q = \gamma P / (B_{ftg} - 2e)$ {factored bearing pressure} [AASHTO 11.6.3.2-1]

[K] WALL REINFORCEMENT DESIGN

1. Determine flexure and shear demands at base of wall for all limit states.

Effect	$F_{L,wall,bot}$	$M_{L,wall,bot}$	SERVICE	STRENGTH		EXTREME				
			SER 01	STR 01-01	STR 01-02	EE 01-01	EE 01-02	EE 01-03	$EE\ 02_{MID}$	$EE\ 02_{END}$
			γ	γ Max	γ Min	γ	γ	γ	γ	γ
#	k / ft	k-ft / ft	#	#	#	#	#	#	#	#
DC	0	0	1.00	1.25	0.90	1.00	1.00	1.00	1.00	1.00
EH_a	0.27	0.31	1.00	1.50	0.90	0.00	0.00	1.00	0.00	0.00
EH_p	0	0	0.00	0.00	0.00	1.00	1.00	1.00	0.00	0.00
$EH_{p,\ CT,\ MID}$	0.84	0.42	0.00	0.00	0.00	0.00	0.00	0.00	1.00	0.00
$EH_{p,\ CT,\ END}$	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.00
EH_{AE}	0.60	0.71	0.00	0.00	0.00	1.00	0.50	0.00	0.00	0.00
IR_{DC}	0	0.48	0.00	0.00	0.00	0.50	1.00	1.00	0.00	0.00
CT_{MID}	-7.91	-21.09	0.00	0.00	0.00	0.00	0.00	0.00	1.00	0.00
CT_{END}	-13.22	-35.24	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.00
$\mathbf{F_{L,bot}}$	-	-	**0.27 k/ft**	**0.40 k/ft**	**0.24 k/ft**	**0.74 k/ft**	**0.57 k/ft**	**0.53 k/ft**	**-7.1 k/ft**	**-13.2 k/ft**
$\mathbf{M_{L,bot}}$	-	-	**0.31 k-ft/ft**	**0.47 k-ft/ft**	**0.28 k-ft/ft**	**0.95 k-ft/ft**	**0.84 k-ft/ft**	**0.8 k-ft/ft**	**-20.7 k/ft**	**-35.2 k/ft**

2. Document wall geometry and reinforcement properties.

- Positive flexure creates tension in the reinforcement closest to the wall face on the embankment side.

- Horizontal reinforcement should be placed inside vertical reinforcement.

c_{clr} = 2.00 in {main reinforcement clear cover} [AASHTO Table 5.10.1-1]

$c_{clr,tie}$ = 1.50 in {cross-tie clear cover}

$c_{clr,tie,\ bot}$ = 3.00 in {cross-tie clear cover at bottom}

$t_{wall,\ min}$	$t_{wall,\ max}$	h_{wall}	L_{wall}	Wall Slope
ft	ft	ft	ft	Run/Rise
0.67	2.38	4.50	1.00	0.19

Reinforcement Properties						
Bar Direction	Description	ψ_b	d_b	A_b	s_b	A_s
#	Text	#	in	in^2	in	in^2/ft
Horizontal	Horiz #5s	5	0.63	0.31	10.00 in	0.37
Stirrups	Vert #5s - Middle	5	0.63	0.31	12.00 in	0.31
Stirrups	Vert #5s - Ends	5	0.63	0.31	6.00 in	0.62

d_s = {effective reinforcement centroid distance from furthest wall face}

3. Check flexure at the Strength and Extreme Event limit states [AASHTO 5.6].

f_y	f'_c	β_1	ε_{cl}	ε_{tl}	ε_{cu}
ksi	ksi	#	#	#	#
60	4.00	0.850	0.002	0.005	0.003

ε_{cl} = {reinforcement compression-controlled strain limit} [AASHTO 5.7.2.1]

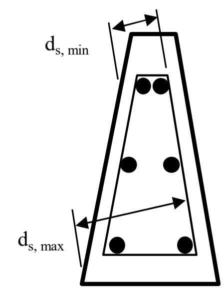
ε_{tl} = {reinforcement tension-controlled strain limit} [AASHTO 5.7.2.1]

ε_{cu} = {unconfined concrete ultimate strain limit} [AASHTO 5.7.2.1]

β_1 = for $f'_c < 4$ksi, $\beta_1 = 0.85$ {compression zone neutral axis ratio} [AASHTO 5.7.2.2]

for $f'_c > 4$ksi, $\beta_1 = 0.85 - 0.05*(f'_c - 4) > 0.65$ksi

Flexure	Location	b	A_s	$d_{s,min}$	$d_{s,max}$
Text	Text	in	in^2/ft	in	in
Positive	Middle	12.00	0.31	6.76	25.55
Negative	Middle	12.00	0.31	6.76	25.55
Negative	Ends	12.00	0.62	6.76	25.55



d_s = {tensile reinforcement centroid distance from wall face}

d_t = {furthest tensile reinforcement distance from wall face}

Limit State	Flexure	c	a	M_n	c / d_s	Check c / d_s Limit	ε_t	φ	**φM_n**
#	Text	in	in	k-ft/ft	#	#	#	#	**k-ft/ft**
STR	Positive	0.54	0.46	39.2	0.033	OK	0.087	0.90	**35.3**
EXT I	Positive	0.54	0.46	39.2	0.021	OK	0.087	1.00	**39.2**
EXT II_{MID}	Negative	0.54	0.46	39.2	0.021	OK	0.087	1.00	**39.2**
EXT II_{END}	Negative	1.07	0.91	77.8	0.042	OK	0.042	1.00	**77.8**

$c = (A_s f_y) / (0.85 f'_c \beta_1 b)$ {distance from extreme compression fiber to neutral axis} [AASHTO 5.6.3.1.1]

$a = \beta_1 c$ {depth of equivalent rectangular stress block} [AASHTO 5.6.2.2]

$M_n = A_s f_y * (d_{s,max} - a / 2)$ [AASHTO 5.7.3.2.2]

$\varepsilon_t = \varepsilon_{cu}*(d_t - c) / c$ {net tensile strength in extreme tension steel at nominal resistance} [strain compatibility]

$\phi = \max(0.75 , \min(0.90 , 0.75 + 0.15*(\varepsilon_t - \varepsilon_{cl}) / (\varepsilon_{tl} - \varepsilon_{cl})))$ [AASHTO C5.5.4.2-1]

Limit State	Flexure	M_u	**D/C**
#	Text	k-ft/ft	#
STR	Positive	0.47	**0.01**
EXT I	Positive	0.95	**0.02**
EXT II_{MID}	Negative	20.66	**0.53**
EXT II_{END}	Negative	35.24	**0.45**

4. Check minimum wall flexural reinforcement [AASHTO 5.6.3.3].

λ = 1.00 {concrete density modification factor} [AASHTO 5.4.2.8]

Limit State	Flexure	f_r	γ_1	γ_2	γ_3	f_{cpe}	S_c	M_{cr}	M_{min}	φM_n	**D/C**
#	Text	ksi	#	#	#	ksi	in^3	k-ft/ft	k-ft/ft	k-ft/ft	#
STR	Positive	0.480	1.60	1.00	0.75	0.00	666	32	0.6	35.3	**0.02**
EXT I	Positive	0.480	1.60	1.00	0.75	0.00	666	32	1.3	39.2	**0.03**
EXT II_{MID}	Negative	0.480	1.60	1.00	0.75	0.00	666	32	27.5	39.2	**0.70**
EXT II_{END}	Negative	0.480	1.60	1.00	0.75	0.00	666	32	32.0	77.8	**0.41**

$f_r = 0.24\lambda\sqrt{(f'_c)}$ {concrete modulus of rupture} [AASHTO 5.4.2.6]

γ_1 = {flexural cracking variability factor} [AASHTO 5.6.3.3]

γ_2 = {prestress variability factor} [AASHTO 5.6.3.3]

γ_3 = {ratio of specified minimum yield strength to ultimate tensile strength of the reinforcement} [AASHTO 5.6.3.3]

f_{cpe} = {compressive stress in concrete due to effective prestress force only} [AASHTO 5.6.3.3]

$S_c = bt_{wall}^2 / 6$ {section modulus per unit length of wall}

5.6.3.3—Minimum Reinforcement

Unless otherwise specified, at any section of a noncompression-controlled flexural component, the amount of prestressed and nonprestressed tensile reinforcement shall be adequate to develop a factored flexural resistance, M_r, greater than or equal to the lesser of the following:

- 1.33 times the factored moment required by the applicable strength load combination specified in Table 3.4.1-1;
- $M_{cr} = \gamma_3\left[\left(\gamma_1 f_r + \gamma_2 f_{cpe}\right)S_c - M_{dnc}\left(\frac{S_c}{S_{nc}} - 1\right)\right]$

5. Check wall crack control at the Service limit state [AASHTO 5.6.7].

$n =$	6.8	$= E_s / E_c$	
$\gamma_e =$	1.00	{class 1 exposure condition}	[AASHTO 5.6.7]

A_s	b	d_s	d_c	ρ	k	j	d_{NA}
in^2/ft	in	in	in	#	#	#	in
0.31	12.00	16.15	1.81	0.0016	0.137	0.954	2.21

$d_c = d - d_s$ {thickness of concrete cover measured from extreme tension fiber to center of flexural reinforcement}

$\rho = A_s / (bd_s)$ {ratio of effective area of tension reinforcement to effective area of concrete}

$k = \sqrt{(2\rho n + (\rho n)^2)} - \rho n$ {ratio of depth of neutral axis to effective depth, d_s}

$j = 1 - k / 3$ {ratio of lever arm of resisting couple to depth, d_s}

$d_{NA} = kd_s$ {depth of netrual axis from extreme compression surface}

$M_{u,SER}$	f_{ss}	β_s	s_{max}	s_{prov}	**D/C**
k-ft/ft	ksi	#	in	in	#
0.31	0.79	1.16	18.00	12.00	**0.67**

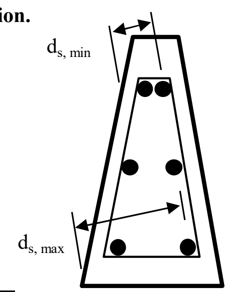
$M_{u,SER} =$ {maximum Service flexural demand per unit length of wall}

$f_{ss} = M_{u,SER} / (A_s j d_s)$

$\beta_s = 1 + d_c / (0.7*(d - d_c))$ {ratio of flexural strain at the extreme tension face to the strain at the centroid of the reinforcement layer nearest the tension face} [AASHTO 5.6.7-2]

$s_{max} = (700\gamma_e) / (\beta_s f_{ss}) - 2d_c$ [AASHTO 5.6.7-1]

6. Check moment capacity to be used for critical wall length in the vehicle impact calculation.



b	# of Bars	A_s	$d_{s,min}$	$d_{s,max}$
in	#	in^2	in	in
54.96	6.00	2.23	6.13	24.92

$d_s =$ {tensile reinforcement centroid distance from wall face}

c	a	M_n	c / d_s	Check c / d_s Limit	ε_t	φ	φM_n
in	in	k-ft	#	#	#	#	**k-ft**
0.84	0.72	169.3	0.054	OK	0.052	0.90	**152.4**

$c =(A_s f_y) / (0.85 f'_c \beta_1 b)$ {distance from extreme compression fiber to neutral axis} [AASHTO 5.6.3.1.1]

$a = \beta_1 c$ {depth of equivalent rectangular stress block} [AASHTO 5.6.2.2]

$M_n = A_s f_y * (d_{s,max} - a / 2)$ [AASHTO 5.7.3.2.2]

$\varepsilon_t = \varepsilon_{cu} *(d_t - c) / c$ {net tensile strength in extreme tension steel at nominal resistance} [strain compatibility]

$\phi = \max (0.75 , \min (0.90 , 0.75 + 0.15*(\varepsilon_t - \varepsilon_{cl}) / (\varepsilon_{tl} - \varepsilon_{cl})))$ [AASHTO C5.5.4.2-1]

7. Check shear design at the Strength, Service, and Extreme limit state [AASHTO 5.7].

$F_{u,STR}$	$F_{u,SER}$	$F_{u,EXT, I}$	$F_{u,EXT, II, MID}$	$F_{u,EXT, II, END}$
k/ft	k/ft	k/ft	k/ft	k/ft
0.40	0.27	0.74	7.06	13.22

b_v	h	d_s	d_v
in	in	in	in
12.00	18.25	16.15	14.54

b_v = {effective width of section measured parallel to neutral axis}

h = {thickness of the section in the direction of loading}

d_s = {depth to the center of flexural reinforcement in the direction of loading}

d_v = MAX(0.72*h, 0.9*d_s) [AASHTO 5.7.2.8]

ϕ_s = 0.9 [AASHTO 5.5.4.2]

Limit State	Flexure	ε	β	θ	V_c	Is Transverse Reinforcement Required?
#	Text	#	#	Degrees	k/ft	
STR	Positive	-	2	45	22.05	Transverse Reinforcement Not Required
SER	Positive	-	2	45	22.05	Transverse Reinforcement Not Required
EXT I	Positive	-	2	45	22.05	Transverse Reinforcement Not Required
EXT II_{MID}	Negative	-	2	45	22.05	Transverse Reinforcement Not Required
EXT II_{END}	Negative	-	2	45	22.05	Transverse Reinforcement Required

ε_s = {net longitudinal tensile strain at the centroid of tension reinforcement} [AASHTO 5.7.3.4.2-4]

β = {factor indicating ability of diagonally cracked concrete to trasmit tension and shear} [AASHTO 5.7.3.4.1]

θ = {angle of inclination of diagonal compressive stresses} [AASHTO 5.7.3.4.1]

$V_c = 0.0316 * \beta * \sqrt{f'_c} * b_v * d_v$ {shear resistance of concrete} [AAHSTO 5.7.3.3-3]

Limit State	Flexure	v_u	s_{max}	**D/C**	$A_{v,\ min}$	**D/C**
#	Text	ksi	in	#	in^2/ft	#
STR	Positive	0.003	11.63	**N/A**	0.15	**N/A**
SER	Positive	0.002	11.63	**N/A**	0.15	**N/A**
EXT I	Positive	0.005	11.63	**N/A**	0.15	**N/A**
EXT II_{MID}	Negative	0.045	11.63	**N/A**	0.15	**N/A**
EXT II_{END}	Negative	0.045	11.63	**0.52**	0.15	**0.24**

$v_u = V_u / (\phi * b_v * d_v)$ {shear stress in concrete} [AASHTO 5.7.2.8-1]

$s_{max} = IF[\ v_u < 0.125 * f'_c\ ,\ MIN(\ 0.8 * d_v\ ,\ 24in\ )\ ,\ MIN(\ 0.4 * d_v\ ,\ 12in\ )]$ {max spacing of shear reinforcement} [AASHTO 5.7.2.6-1/2]

$A_{v,\ min} = 0.0316 * \sqrt{f'_c} * b_v * s / f_y$ {minimum area of transverse reinforcement} [AASHTO 5.7.2.5-1]

Limit State	Flexure	V_s	V_n	ϕV_n	**D/C**
#	Text	k/ft	k/ft	k/ft	#
STR	Positive	22.23	44.28	39.86	**0.01**
SER	Positive	22.23	44.28	39.86	**0.01**
EXT I	Positive	21.99	44.04	39.64	**0.02**
EXT II_{MID}	Negative	10.67	32.72	29.45	**0.24**
EXT II_{END}	Negative	24.65	46.70	42.03	**0.31**

$V_s = A_v * f_y * d_v * \cot(\theta) / s$ {shear resistance of transverse reinforcement} [AASHTO 5.7.3.3-4]

$V_n = MIN(\ 0.25 * f'_c * b_v * d_v\ ,\ V_c + V_s\ )$ {nominal shear resistance} [AASHTO 5.7.3.3-1/2]

RETAINING WALL 06.50L

3.0 – Wall Cap with Fall Protection Fence Design

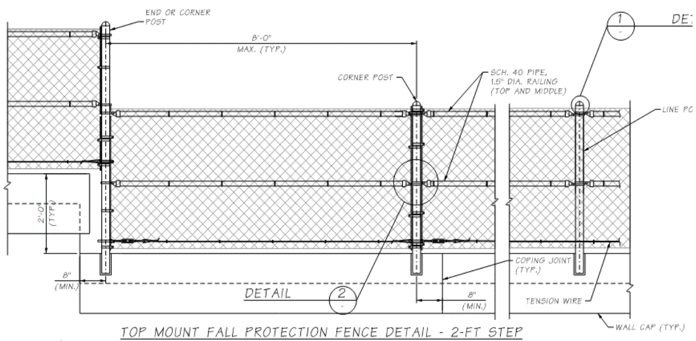
[A] BASIS & ASSUMPTIONS

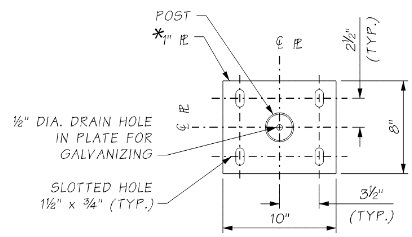
-To check the Fall Protection Fence Design for walls 6.50L, 6.63L, 6.99L, 7.37L, and 7.45L
-Applicable loading: wind loads (per AASHTO LRFD) and fall protection loads (per WSDOT RDM)

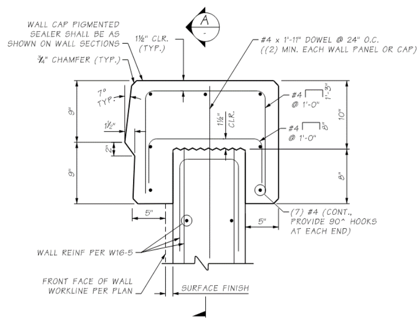
[B] REFERENCES

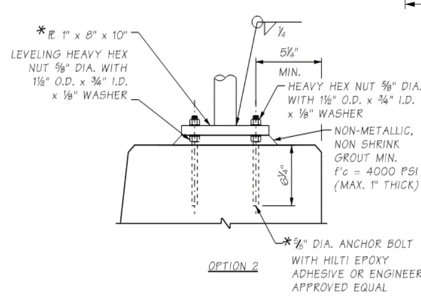
Acronym	Source
AASHTO	AASHTO LRFD 2017 8th Edition
BDM	WSDOT Bridge Design Manual M23-50.18, June 2018
RDM	WSDOT Roadway Design Manual M22-01.15, July 2018

[C] GEOMETRY AND MATERIAL PROPERTIES [REFERENCE]









Post & Railing

H_{max} =	4.00 ft	{maximum post height}
S_{max} =	8.00 ft	{maximum post spacing}
N_{hor} =	3	{number of horizontal elements}
h_1 =	0.25 ft	{bottom hor. element height}
h_2 =	1.75 ft	{middle hor. element height}
h_3 =	3.50 ft	{top hor. element height}

Anchorage

Bolt size =	0.6 in	
N_{bolts} =	4	{number of bolts}
d_{edge} =	1.50 in	{edge distance}
s_1 =	7.00 in	{bolt spacing}
s_2 =	5.00 in	{bolt spacing}
t_{plate} =	1.00 in	{plate thickness}
b_{plate} =	10.00 in	{plate width}
l_{plate} =	8.00 in	{plate length|

Chain Link

b_{open} =	2.00 in	{fabric maximum opening}
WG =	11 gauge	{fabric wire gauge}
Coating =	Vinyl (black)	

Cap Concrete & Reinforcement

f'_c =	4.00 ksi	{concrete strength}
f_y =	60.00 ksi	{rebar yield strength}
d_{dow} =	#4	{dowel size}
n_{dow} =	2	{number of dowels}

[D] FALL PROTECTION LOADS

- Apply FP as a Live Load in design load combinations

P_{fp} = 200.00 lb {point load in any direction at top of post} [RDM M22-01 730.04(7)b]

w_{fp} = 50.00 plf {pedestrian LL distributed load} [AASHTO 13.8.2]

$V_{LL} = P_{fp} + w_{fp}*L$ [AASHTO 13.8.2-1]

y = 3.50 ft {height above bottom of fence}

P_{FP} = 600.00 lb {fall protection loading per post}

V_{LL} = 0.60 k per post

M_{LL} = 2.10 k-ft per post

[E] WIND LOADS ON FENCING

AASHTO LRFD Bridge Design Spec. - 9th Ed. AASHTO for Chain Link Fence (13.8.2)

P_d = 15.00 psf [AASHTO 13.8.2]

P_{WS} = 480.00 lb {wind loading per post}

V_{WS} = 0.48 k per post

M_{WS} = 0.96 k-ft per post

[F] COMBINED LOAD DEMAND ON POSTS

Load Combo	LL	WS
STR III	0.00	1.00
STR V	1.35	1.00

Load	STR III		STR V	
	Shear, V	Bending, M	Shear, V	Bending, M
LL	0.00 k	0.00 k-ft	0.81 k	2.84 k-ft
WS	0.48 k	0.96 k-ft	0.48 k	0.96 k-ft
Total	**0.48 k**	**0.96 k-ft**	**0.81 k**	**2.84 k-ft**

Min. Z_x for Pipe $\geq$ 1.08 in³

[G] POST SECTIONAL CAPACITY

-Schedule 80 Pipe 2" Diameter for Line Posts

-Schedule 80 Pipe 2 1/2" Diameter for End or Corner Posts

Shear Capacity

ϕ_v = 1.00 [AASHTO 6.5.4.2]

$V_n = 0.5\ F_{cr}\ A_g$ [AASHTO 6.12.1.2.3c-1]

$F_{cr,V}$ = larger of: $1.60\ E / \sqrt{(L_v/D)}\ (D/t)^{1.25}$ [AASHTO 6.12.1.2.3c-2]

$0.78\ E / (D/t)^{1.5}$ [AASHTO 6.12.1.2.3c-3]

$F_{cr,V}$ shall not exceed: $0.58\ F_y$

Moment Capacity

ϕ_b = 1.00 [AASHTO 6.5.4.2]

Where $D/t \leq 0.45\ E/F_y$ [AASHTO 6.12.2.2.3]

For Yielding, $M_n = Z_x F_y$ [AASHTO 6.12.2.2.3-1]

For Local Buckling, $M_n = S_x F_{cr}$ Members are not slender or non-compact, Local Buckling does not apply

	Schedule 80 Pipe		
Shape	2"	2 $^1/_2$"	
$F_{cr,V}$ =	20.3	20.3	
F_y =	35.00 ksi	35.00 ksi	
D = O.D. =	2.38 in	2.88 in	
I.D. =	1.94 in	2.32 in	
Design Wall Thickness, t =	0.20 in	0.26 in	
X-Sect. Area, A_g =	1.39 in²	2.11 in²	
S_x =	0.70 in³	1.72 in³	
Z_x =	0.96 in³	1.44 in³	
D/t =	11.64	11.19	
0.45 E/F_y =	372.86	372.86	
0.07 E/F_y =	58.00	58.00	local buckling does not apply

V_n =	14.11 k	21.42 k
$\phi_v V_n$ =	14.11 k	21.42 k
d/c =	0.06	0.04
	OK	OK

M_n =	2.81 k-ft	4.20 k-ft
$\phi_b M_n$ =	2.81 k-ft	4.20 k-ft
d/c =	1.01	0.68
	SAY OK	OK

* Schedule 80 Pipe may be used if post spacing is 8' or less or fence height is 4' or less

[H] COMBINED LOAD DEMAND ON ANCHOR BOLTS

Load	STR III		STR V	
	Shear, V	Tension, T	Shear, V	Tension, T
Total	**0.12 k**	**0.93 k**	**0.20 k**	**2.75 k**

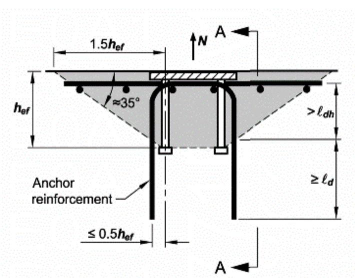
[I] ANCHOR CAPACITY

-Use ACI 318-14 Section 17.4.5 to calculate the bond strength of anchors in tension.

$$N_{cb} = \left(\frac{A_{Nc}}{A_{Nco}}\right)\psi_{ec,N}\psi_{ed,N}\psi_{c,N}\psi_{cp,N}N_b$$

h_{ef} =	6.25 in	{embed depth}
n =	1	{number of anchors in tension}
A_{Nco} =	351.56 in²	{area of influence}
A_{Nc} =	351.56 in²	{area of influence per bolt}
k_c =	24	
N_b =	23.72 k	
$\psi_{ec,N}$ =	1	
$\psi_{ed,N}$ =	0.748	
$\psi_{cp,N}$ =	1	
φ =	0.75	
N_{cb} =	13.31 k	{concrete blockout strength per bolt}
Check =	OK	

If anchor reinforcement is developed on both sides of the breakout surface, the design strength of the anchor reinforcement shall be permitted to be used instead of the breakout strength determining ϕN_n.



Elevation

s_{stir} =	12.00 in	{stirrup spacing}
s_1 =	7.00 in	{bolt spacing}
diff =	2.50 in	{distance from bolt to reinf}
0.5 h_{ef} =	3.13 in	

Anchor reinforcement design strength to be used

Stirrups =	2- #4
A_s =	0.39 in²
φ =	0.75
ϕN_n =	17.67 k

PARSONS		MADE BY	DATE	CHK BY	DATE
		K. GARCIA	11/22/2021	N. ALA	1/22/2021
Job Number	WBS Number	TITLE	I-405 Renton to Bellevue		
650512	00531		Wall Cap Design		

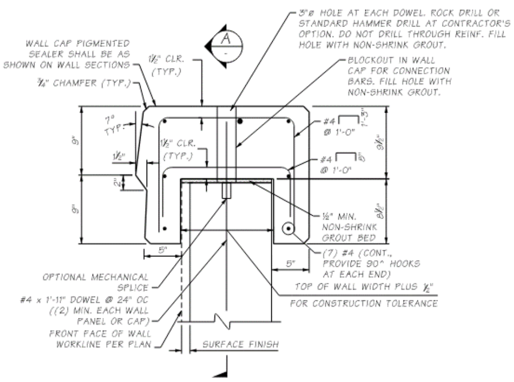
[A] BASIS

To check the wall cap reinforcement design.

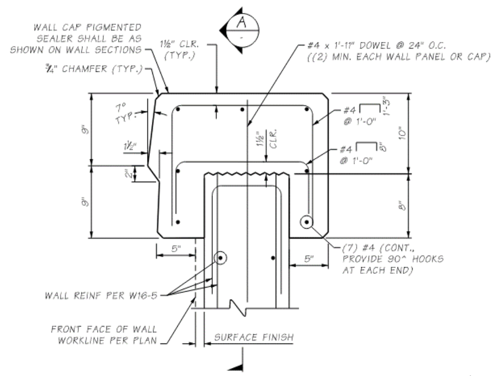
[B] REFERENCES

Acronym	Source
AASHTO	AASHTO LRFD 2017 8th Edition
BDM	WSDOT Bridge Design Manual M23-50.18, June 2018
RDM	WSDOT Roadway Design Manual M22-01.15, July 2018

[C] GEOMETRY AND MATERIAL PROPERTIES



PRECAST WALL CAP DETAIL



OPTIONAL CIP WALL CAP DETAIL

½" CH.

t_{wall} =	14.50 in	{wall thickness}
d_{dowel} =	7.25 in	{depth to dowel}

Cap Concrete & Reinforcement

f'_c =	4.00 ksi	{concrete strength}	
f_y =	60.00 ksi	{rebar yield strength}	
ϕ_f =	0.90	{flexural resistance factor}	[AASHTO 5.5.4.2]

[D] FENCE LOADING

Fall Protection

P_{FP} = 200.00 lb {point load in any direction at top of post} [RDM M22-01 730.04(7)b]
w_{fp} = 50.00 plf {pedestrian LL distributed load} [AASHTO 13.8.2]
$V_{LL} = P_{fp} + w_{fp}*L$

P_{FP} = 600.00 lb {fall protection loading per post}
V_u = 0.60 k per post
h = 4.33 ft {top of fence to wall-cap interface}
M_u = 2.60 k-ft per post

Wind

P_d = 15.00 psf [AASHTO 13.8.2]
P_{WS} = 480.00 lb {wind loading per post}
V_{WS} = 0.48 k per post
M_{WS} = 1.36 k-ft per post

Load Combo	LL	WS
STR III	0.00	1.00
STR V	1.35	1.00

Load	STR III		STR V	
	Shear, V	Bending, M	Shear, V	Bending, M
LL	0.00 k	0.00 k-ft	0.81 k	3.51 k-ft
WS	0.48 k	1.36 k-ft	0.48 k	1.36 k-ft
Total	**0.48 k**	**1.36 k-ft**	**0.81 k**	**3.51 k-ft**

[E] CHECK FLEXURE

M_u = 42.12 k-in
$A_{s,req}$ = 0.109 in²

Bar size = #4
s = 24.0 in Bar Spacing
$A_{s,prov}$ = 0.200 in²/ft Provided Area of Steel
a = 0.294 in $(A_s f_y)/(0.85f'_c b)$
β_1 = 0.85 in
c = 0.346 in a/β_1
d_e = 7.25 in
M_n = 7.10 k-ft/ft $(A_s f_y)(d - a/2)$

ε_{tl} = 0.0050
ε_c = 0.0030
ε_{cl} = 0.0020 [AASHTO 5.7.2.1]
ε_t = 0.0599 $\epsilon_c(d_e - c)/c$ [AASHTO 5.5.4.2.1]
Tension-Controlled

[F] CHECK INTERFACE SHEAR

$V_{ni} = cA_{cv} + \mu(A_{vf}f_y + P_c)$ [AASHTO 5.7.4.3-3]

$V_{ni} \leq K_1 f_c' A_{cv}$ [AASHTO 5.7.4.3-4]

$V_{ni} \leq K_2 A_{cv}$ [AASHTO 5.7.4.3-5]

b_{vi} =	24.00 in	{effective length}
L_{vi} =	14.50 in	{wall thickness}
A_{cv} =	348.00 in²	{shear interface area}

For concrete placed against a clean concrete surface, free of laitance, but not intentionally roughened: [AASHTO 5.7.4.4]

c =	0.075 ksi	{cohesion factor}
u =	0.60	{friction factor}
K_1 =	0.20	
K_2 =	0.80 ksi	
P_c =	0.00 k	{conservatively assume no axial load}

V_{ni} =	14.40 k	{conservatively assume no concrete contribution}
D/C =	0.06	OK

RETAINING WALL 06.50L

4.0 – Bridge 405-24 Bearing Pressure Calculation

[A] BASIS

- To calculate bearing pressure under existing pier 1 abutment of Bridge 405-24

[B] REFERENCES

Acronym	Source
AASHTO	AASHTO LRFD Bridge Design Specifications 8th Edition
BDM	WSDOT Bridge Design Manual (M23-50.18) - June 2018
As-Builts	NE30th Bridge- As Built Drawings
GEM	Wall Package 1 - Retaining Wall 6.50L Geotechnical Design Memo, Oct 2021

[C] DESIGN NOTES AND PARAMETERS

[D] MATERIAL PROPERTIES

γ_c =	145.0 pcf	{plain concrete unit weight for loads and models}	[BDM Table 3.8.1]
γ_{rc} =	155.0 pcf	{reinforced concrete unit weight for loads and models}	[BDM Table 3.8.1]
γ_{ps} =	165.0 pcf	{prestressed concrete unit weight for loads and models}	[BDM Table 3.8.1]

Concrete									
Elements	f'_c	K_1	E_c	f'_{ce}	E_{ce}	α_{TU}	υ	G_c	G_{ce}
Text	ksi	#	ksi	ksi	ksi	$°F^{-1}$	#	ksi	ksi
Concrete	4.00	1.00	4555	5.20	4967	6.00E-06	0.20	1898	2070

f'_c = {concrete compressive strength }

K_1 = {correction factor for source of aggregate}

$E_{rc} = 120000\ K_1(\gamma_{rc})^2(f'_c)^{0.33}$ {concrete modulus of elasticity} [AASHTO 5.4.2.4]

$f'_{c,e} = 1.3f'_c$ {expected concrete compressive strength} [SGS 8.4.4-1]

$E_{rc,e} = 120000\ K_1(\gamma_{rc})^2(f'_{c,e})^{0.33}$ {expected concrete modulus of elasticity} [AASHTO 5.4.2.4]

α_{TU} = {coefficient of thermal expansion} [DCM 8.4.2.I.3]

υ = {poisson's ratio} [AASHTO 5.4.2.5]

$G_c = E_{rc} / (2*(1+\upsilon))$ {concrete shear modulus}

$G_{ce} = E_{rc,e} / (2*(1+\upsilon))$ {concrete expected shear modulus}

ASTM A706 Grade 60 Reinforcing Steel							
Bar Size	f_y	f_u	E_s	f_{ye}	f_{ue}	ε_y	ε_{ye}
#	ksi	ksi	ksi	ksi	ksi	#	#
All	60	80	29000	68	95	0.0021	0.0023

f_y = {minimum yield strength} [ASTM A706-16 Table A1.2]

f_u = {minimum tensile strength} [ASTM A706-16 Table A1.2]

E_s = {steel reinforcement modulus of elasticity} [AASHTO 5.4.3.2]

f_{ye} = {expected minimum yield strength} [SGS Table 8.4.2-1]

f_{ue} = {expected minimum tensile strength} [SGS Table 8.4.2-1]

$\varepsilon_y = f_y / E_s$ {nominal yield strain}

ε_{ye} = {expected yield strain} [SGS Table 8.4.2-1]

back soil properties										
Soil Type	γ_s	ϕ_{soil}	C	K_a	K_0	K_p	K_h	K_{AE}	ϕ_{soil} for COF	tan δ = tan ϕ
#	pcf	deg	psf	#	#	#	#	#	deg	#
Fill	125.0	36	0.00	0.350	-	6.00	0.25	0.44	36.00	0.58

entered as increment

γ_s = {soil unit weight}

ϕ_{soil} = {internal friction angle of soil}

C= {Soil cohession}

K_a = {active soil pressure coefficient} = (1-sin(ϕ))/(1+sin(ϕ))

K_0 = {at rest soil pressure coefficient} = 1-sin(ϕ)

K_p = {active soil pressure coefficient} = (1+sin(ϕ))/(1-sin(ϕ))

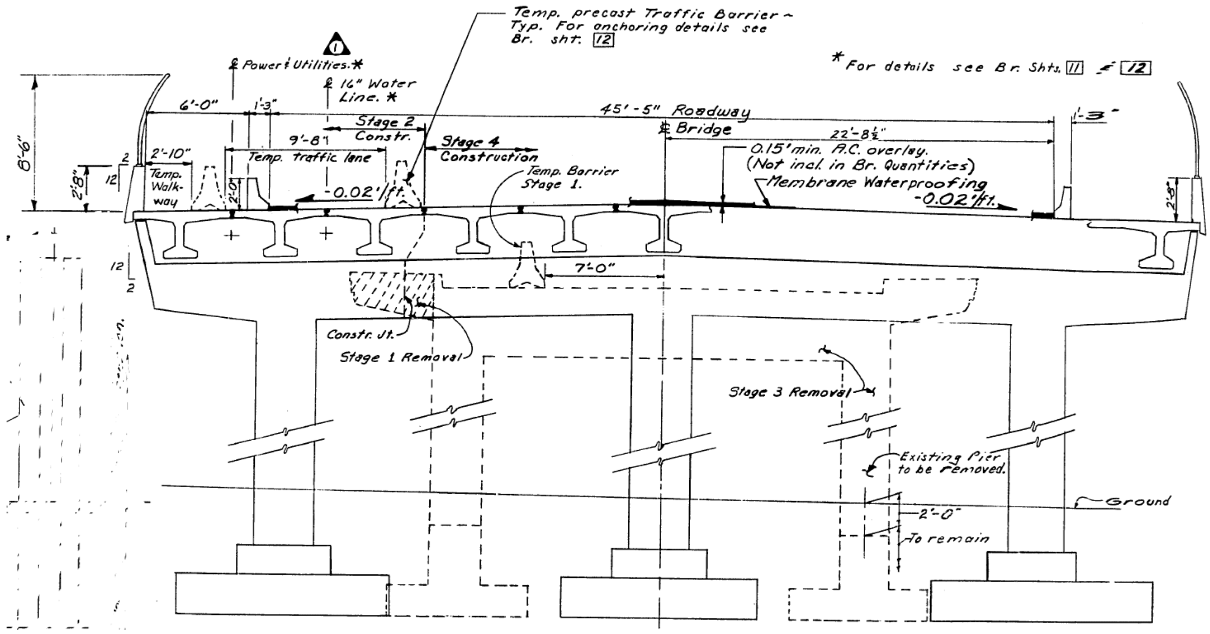
K_h = {horizontal seismic acceleration coefficient} - taken 0.5 × PGA [AASHTO A11.3.1, AASHTO 11.6.

K_{AE} = {seismic active earth pressure coefficient} [AASHTO A11.3.1]

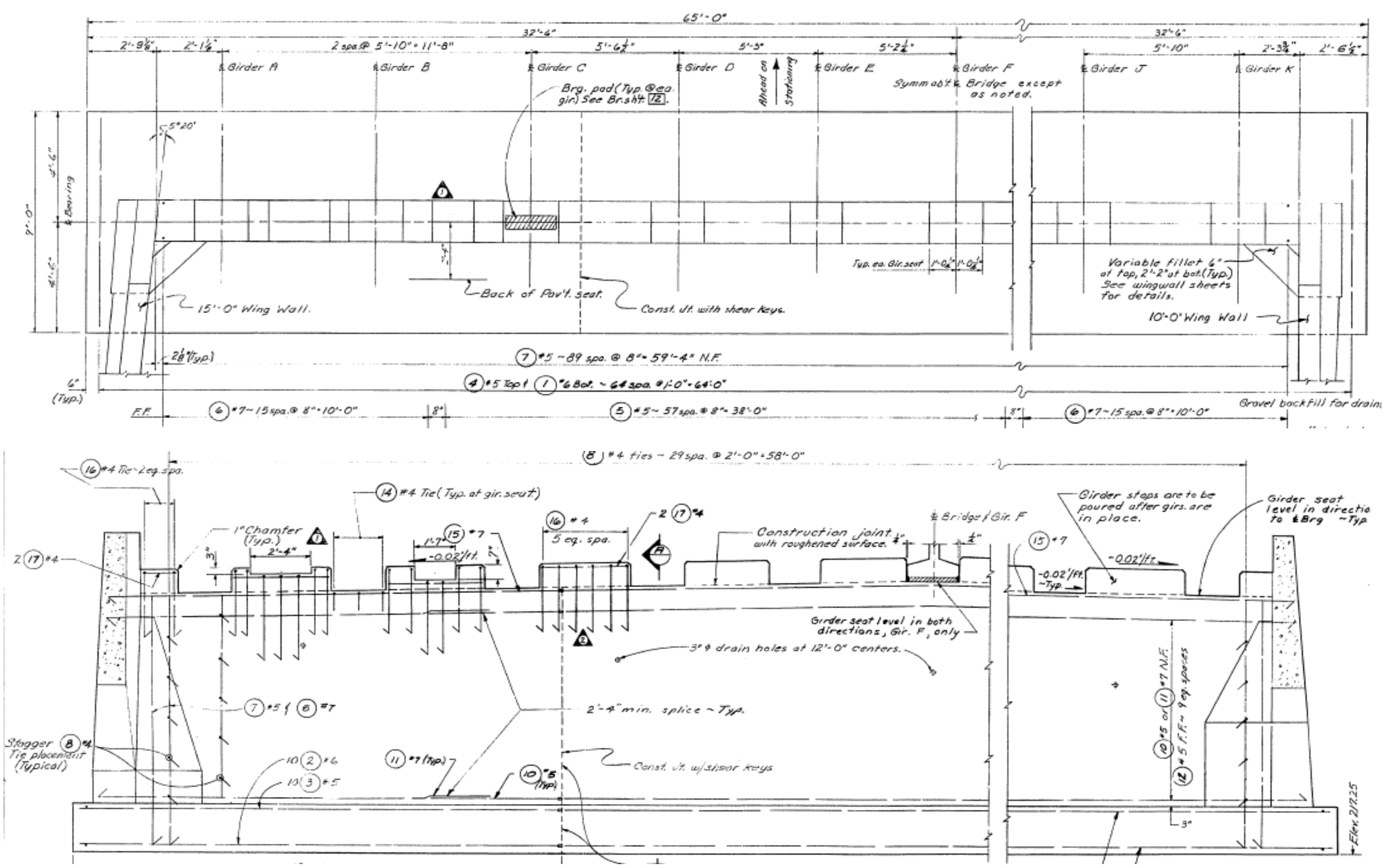
tan δ = {Coefficient of friction between soil and bottom of footing} = tan φ for cast in place concrete against soil [BDM 7.7.4 C]

[E] GEOMETRY

Bridge Typical Section:

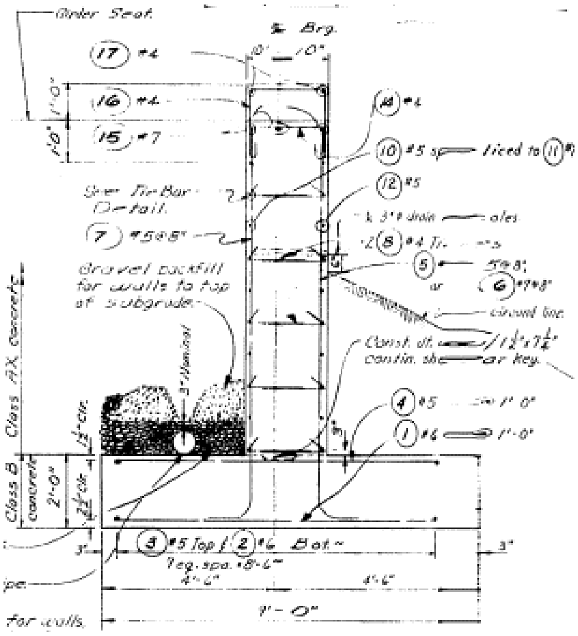


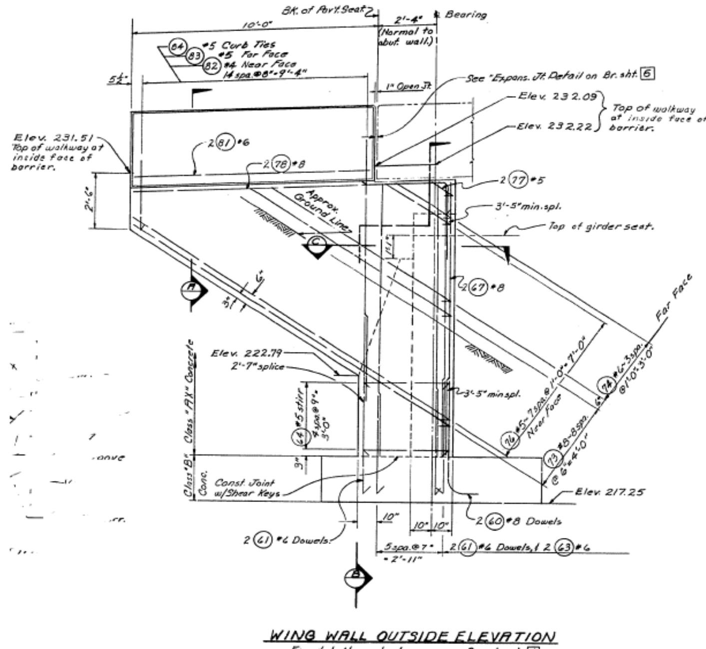
Pier 1 Abutment:

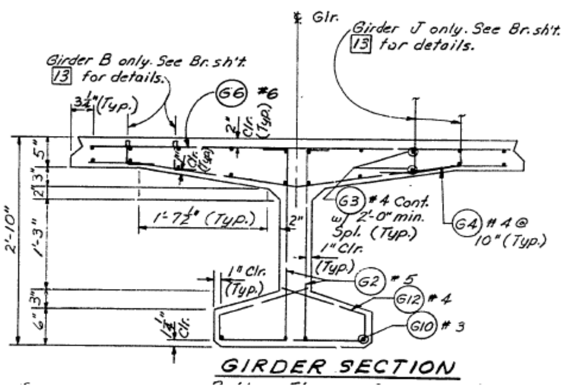


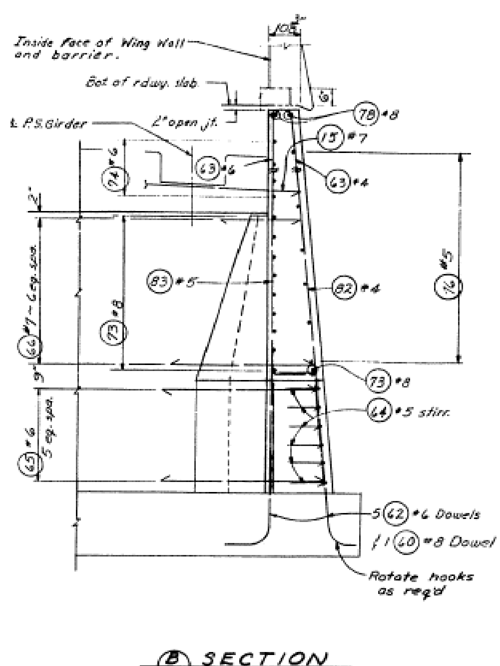
GIRDER SEAT ELEVATIONS

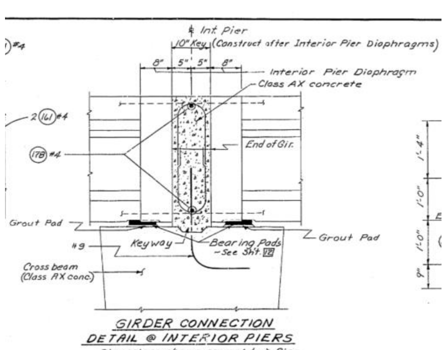
Girder	A	B	C	D	E	F	G	H	I	J	K
Pier 1	229.35	229.47	229.59	229.70	229.80	229.85	229.80	229.70	229.59	229.47	229.35











[F] LOAD CALCULATION

Span length	Girder length	no of girders	girder area	total SS area	no of large barriers	no of small barriers	area of large barrier	area of small barrier	weigh of large barrier	weigh of small barrier	assume weigh of throw fence
ft	ft	#	sq.ft	sq.ft	#	#	sq.ft	sq.ft	klf	klf	klf
78.50	77.06	11	4.58	55.56	2	2	3.09	1.77	0.96	0.55	0.20

for two barriers, two throw fences

abutment footing length	abutment footing width	abutment footing thickness	abutment stem thickness	abutment stem avg. height	abutment stem area	abutment weight
ft	ft	ft	ft	sq.ft	sq.ft	kip
65.00	9.00	2.00	1.67	10.23	651.75	349.72

assume utility weight
klf
0.20

overlay thickness	overlay weight
ft	klf
0.15	1.59

wingwall thickness	10ft wingwall area	15ft wingwall area	10ft abutment weight	15ft abutment weight
ft	sq.ft	sq.ft	kip	kip
1.38	95.82	140.42	39.60	44.39

Girder seat elevation		
A	229.35	229.48
B	229.47	219.25
C	229.59	**10.23**
D	229.7	
E	229.8	
F	229.85	
G	229.8	
H	229.7	
I	229.59	
J	229.47	
K	229.35	

end diaphragm				intermediate diaphragm			
diaph. length	diaph. width	diaph. thickness	diaph. weight	diaph. length	diaph. width	diaph. thickness	diaph. weight
ft	ft	ft	kip	ft	ft	ft	kip
59.70	1.38	4.08	51.95	54.93	0.67	1.92	10.88

total DC					DW	
SS	barriers	abutment	wingwalls	diaph.	overlay	utilities
kip	kip	kip	kip	kip	kip	kip
353.23	50.21	349.72	83.99	56.03	46.79	5.89

total roadway width	total sidewalk width	no lanes	bearing reaction - HL93 + IM	bearing reaction - Ped
ft	ft	#	kip	kip
45.42	12.00	3	265.12	27.55

note: since structure is continuous over the intermediate piers, 0.39 coefficient is used to determine reactions for barriers, utilities, intermediate diaphragm, overlay, HL93 Lane and pedestrian load. For DC from superstructure and live load from HL93 truck simple span is assumed.

Bearing Slip Factor: 20%

Bearing pad&grout thickness: 0.125 ft

Bearing Type: Elastomeric

[BDM 7.5.6 A]

EQ Normal to abut. Force	Application height	Eccentricity CL Ftg	Moment CL Ftg
kip	ft	ft	k-ft
247.71	12.23	0.000	3029.8

Eccentricity: Positive for reaction right of footing CL (positive for reaction on bridge side)

approach slab DC							
Element	Span	Length (along wall)	Thickness	Volume	Weight	Eccentricity CL Ftg	Moment CL Ftg
Text	ft	ft	ft	cu.ft	kip	ft	k-ft
Approach Slab DC	0.0	0.0	0.00	0.0	122.50	-2.62	-321.56

Per BDM 7.5.4 B, approach slab DL is 2kip/ft

soil Weight							
Element	Length	Width	Thickness	Volume	Weight	Eccentricity CL Ftg	Moment CL Ftg
Text	ft	ft	ft	cu.ft	kip	ft	k-ft
EV_{heel}	65.0	3.67	4.44	1058.3	132.3	-2.67	-352.81
EV_{toe}	65.0	3.67	3.36	800.9	100.1	2.67	266.99

lateral effects for stability						
Case	Length	Height	Force	Application height	Moment CL Ftg	
Text	ft	ft	kip	ft	k-ft	depth for passive pressure calculation = 2.00 ft
EH_a	59.70	6.44	54.2	2.15	116.3	
LS	0.0 *	6.44	0.0	3.22	0.0	Equivalent Soil Height for LS = 4.00 ft [AASHTO 3.11.6.4]
P_{AE}	59.7	6.44	190.34	-	554.76	
R_{ep}	0.0 **	3.4	0.0	1.12	0.0	{length is set to zero to ignore effect of passive soil pressure in calculatio
R_T	59.7	-	694.4	0.00	0.0	{soil on toe weight excluded from friction force calculation}
P_{IR} - Normal	-	-	166.5	4.03	671.6	{includes inertial soil mass}
P_{seis} - Stability	-	-	261.7	-	948.9	$\{P_{AE} + 0.5P_{IR}\}$; used for bearing, overturning and : [AASHTO 11.6.5.1] per AASHTO 11.6.3.1

* Per BDM 7.5.4 B, since there is approach slab, LS is set to zero

** Per BDM 7.5.4 C, passive resistance is neglected

[G] LOAD FACTORS

Strength I, Service I, and Extreme I are the load combinations that affect the design of abutment.

Strength Cases with wind on structure, or wind on live load, STR II or STR V, will not control design as these loads are in direction parallel to abutment.

Load Factors											
Limit State	DC max	DC min	DW max	DW min	EH max	EH min	EV max	EV min	CR & SH	LL/LS	EQ
Strength I	1.25	0.90	1.50	0.65	1.50 Active 0.90 Passive	0.90 Active 0.90 Passive	1.35	1.00	0.50	1.75	0.00
Service I	1.00	-	1.00	-	1.00	-	1.00	-	1.00	1.00	0.00
Extreme Event I	1.00	-	1.00	-	1.00	-	1.00	-	1.00	0.50	1.00
Construction	1.00	0.90	-	-	1.50 Active	-	-	1.00	-	1.00	-

Load Factors - Continued			
Limit State	TU max	TU min	BR
Strength I	0.50	0.50	1.75
Service I	1.00	-	1.00
Extreme Event I	-	-	0.50
Construction	-	-	-

[H] CHECK BEARING STRESS

- Per AASHTO LRFD 11.6.3.3, for foundations on soil, the location of the resultant of the reaction forces shall be within the middle two-thirds of the base width.
- Per AASHTO Guide Specs for LRFD Seismic Bridge Design, if full Live Load is present, the resultant should be within the middle eight-tenth of the base.

  The resultant of the reaction forces shall be within the middle 2/3 if no LL

Limit State	R_u	R_{ep}	M_o	e_o	X_o	σ_v
	kip	k-ft	k-ft	ft	ft	ksf
Strength I - max	2175	0	-1530	0.70	3.80	4.41
Strength I - min	1693	0	-1300	0.77	3.73	3.49
Service I	1593	0	-1992	1.25	3.25	3.77
Extreme I	1447	0	-4002	2.77	1.73	6.42

R = {Resultant force, R_u ultimate activating force, R_{ep} passive force}

M_o = {Overturning Moment about CL of footing}

e = {Eccentricity} = M/R

X = {Resultant location} = B/2 - e

σ = {Maximum footing pressure on soil with a uniform distribution} = R / 2X

[I] CHECK SLIDING

Resistance factor for Strength/friction, ϕ_T =	0.80	[AASHTO 10.5.5.2.2]
Resistance factor for Extreme/friction, ϕ_T =	1.00	[AASHTO 10.5.5.3.3]
Resistance factor for Strength/passive, ϕ_{ep} =	0.50	[AASHTO 10.5.5.2.2]
Resistance factor for Extreme/passive, ϕ_{ep} =	1.00	[AASHTO 10.5.5.3.3]

Check Sliding: Normal to abutment

Limit State		$\phi_T Q_T$	$\phi_{ep} Q_{ep}$	Q_R *	factored sliding load	**D / C**	
		kip	kip	kip	kip	#	
Strength I	max	773	0	773	81	**0.105**	{Note: LL from Super not included in sliding resistance}
	min	549	0	549	81	**0.148**	{Note: LL from Super not included in sliding resistance}
Service I		756	0	756	54	**0.072**	{Note: LL from Super not included in sliding resistance}
Extreme Event I		756	0	756	452	**0.598**	{Note: LL from Super not included in sliding resistance}

* Passive earth pressure is neglected

Q_R = {factored sliding resistance} = $\phi_T Q_T + \phi_{ep} Q_{ep}$

$\phi_T Q_T$ = {frictional component of sliding resistance}

ϕepQep = {passice earth pressure component of sliding resistance}

RETAINING WALL 06.50L

5.0 – Bridge 405-24 Monitoring Threshold

PARSONS		MADE BY	DATE	CHK BY	DATE
		N. Ala	12/02/2021	E. Kelley	12/03/2021
Job Number	WBS Number	TITLE	WSDOT - Renton to Bellevue		
650512	00531		Bridge 405-24 Monitoring Threshold		

[A] BASIS

- To calculate thresholds specified for monitoring of Bridge 405-24 (N 30th ST Bridge) during soil nail construction

[B] REFERENCES

Acronym	Source
AASHTO	AASHTO LRFD Bridge Design Specifications 8th Edition
BDM	WSDOT Bridge Design Manual 2018
As-Builts	NE30th Bridge- As Built Drawings
GEM	Wall Package 1 - Retaining Wall 6.50L Geotechnical Design Memo, Oct 2021
INSP.	Bridge 405/24 Inspection Report - 6/18/2018
RTG.	Bridge 405/24 Load Rating Summary 08/93

[C] DESIGN NOTES AND PARAMETERS

1. In order to determine thresholds, load rating equations/method is used. A settlement in abutment 1 is calculated at which the legal truck load rating factor for negative moment at pier 2 becomes 1.00.
From [RTG.] the controlling legal truck is AASHTO 2 (TYPE 3S2) hence that truck is used in this calculation.
2. It is figured that because of bridge construction sequence barriers and self weight of girders do not contribute in negative moment at the interior piers which is consistent with the [RTG.] values provided. Bridge diaphragms do not contribute in negative moment at the interior piers per the as-builts.
3. Overlay and utilities contribute in negative moment at interior piers.
4. A CSi Bridge model of one girder line of the sturcture is created and used in this analysis.

[D] MATERIAL PROPERTIES

γ_c =	145.0 pcf	{plain concrete unit weight for loads and models}	[BDM Table 3.8.1]
γ_{rc} =	155.0 pcf	{reinforced concrete unit weight for loads and models}	[BDM Table 3.8.1]
γ_c =	140.0 pcf	{AC overlay}	[BDM Table 3.8.1]

Plain Concrete				
Elements	f'_c	K_1	E_c	υ
Text	ksi	#	ksi	#
Typ	4.00	1.00	4555	0.20
Deck Bulb T	7.00	1.00	5479	0.20

f'_c = {concrete compressive strength }

K_1 = {correction factor for source of aggregate}

$E_{rc} = 120000\ K_1(\gamma_{rc})^2(f'_c)^{0.33}$ {concrete modulus of elasticity} [AASHTO 5.4.2.4]

υ = {poisson's ratio} [AASHTO 5.4.2.5]

ASTM A706 Grade 60 Reinforcing Steel				
Bar Size	f_y	f_u	E_s	ε_y
#	ksi	ksi	ksi	#
All	60	80	29000	0.0021

f_y = {minimum yield strength} [ASTM A706-16 Table A1.2]

f_u = {minimum tensile strength} [ASTM A706-16 Table A1.2]

E_s = {steel reinforcement modulus of elasticity} [AASHTO 5.4.3.2]

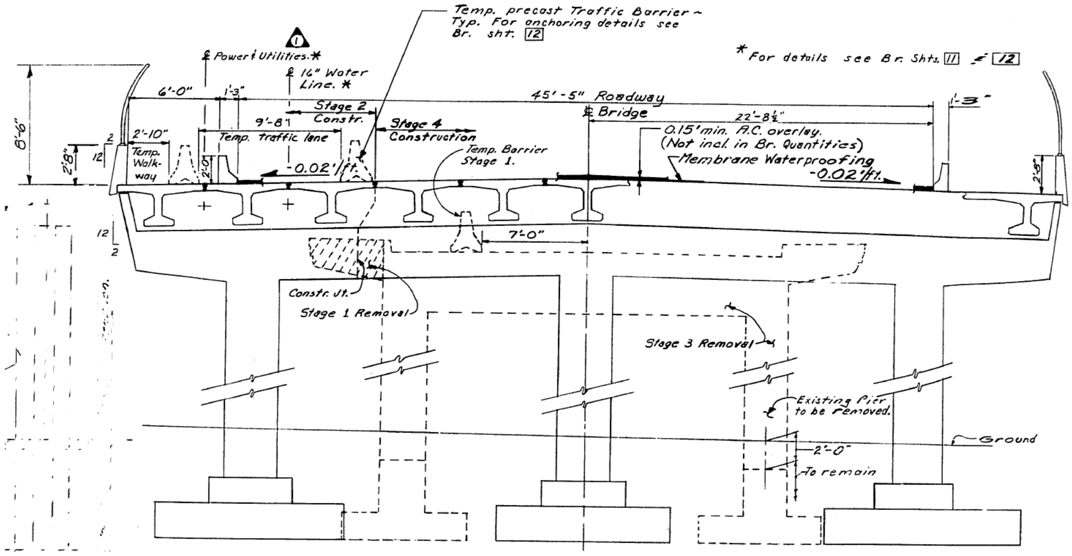
$\varepsilon_y = f_y / E_s$ {nominal yield strain}

ε_{tl} =	0.005	{tension-controlled reinf. steel strain limit}	[AASHTO 5.6.2.1]
ε_{cl} =	0.002	{compression-controlled reinf. steel strain limit}	[AASHTO 5.6.2.1]
ε_c =	0.003	{maximum usable concrete compression strain}	[AASHTO 5.6.2.1]

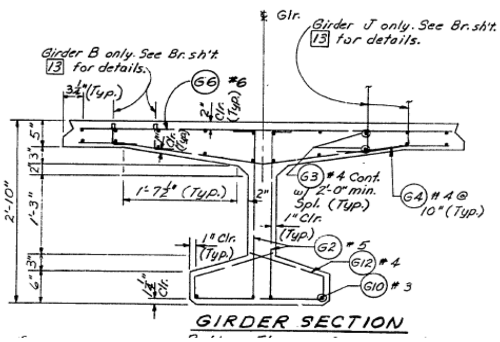
PARSONS		MADE BY	DATE	CHK BY	DATE
		N. Ala	12/02/2021	E. Kelley	12/03/2021
Job Number	WBS Number	TITLE	WSDOT - Renton to Bellevue		
650512	00531		Bridge 405-24 Monitoring Threshold		

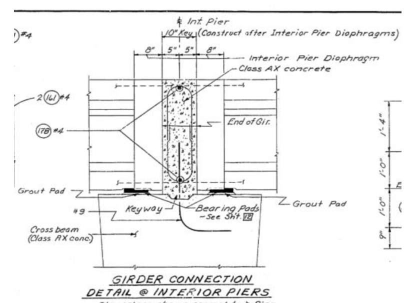
[E] GEOMETRY

Bridge Typical Section:

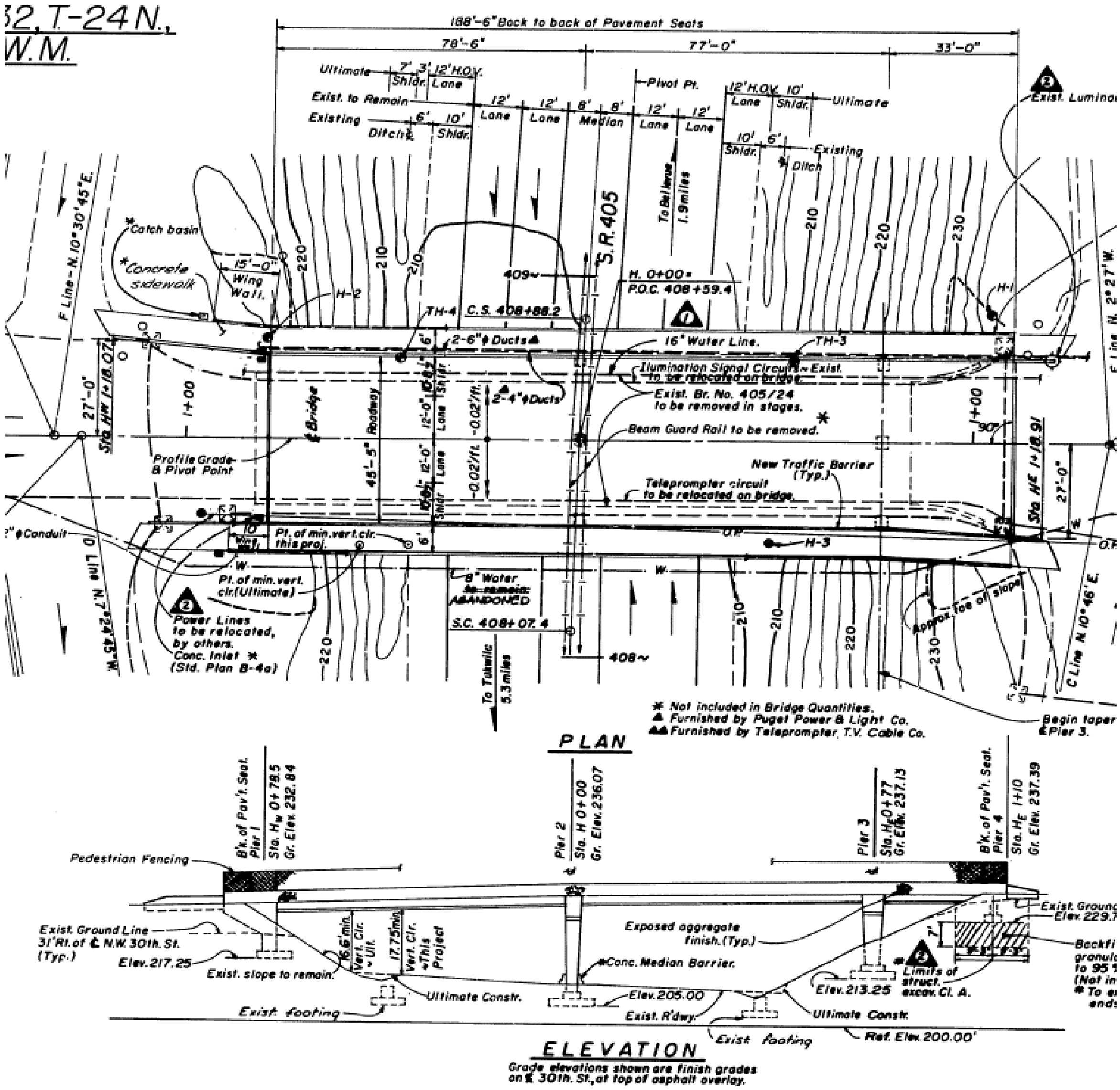


Bridge girder and closure joint at piers





Bridge Layout:



PARSONS		MADE BY	DATE	CHK BY	DATE
		N. Ala	12/02/2021	E. Kelley	12/03/2021
Job Number	WBS Number	TITLE	WSDOT - Renton to Bellevue		
650512	00531		Bridge 405-24 Monitoring Threshold		

[F] CALCULATE LLDF

From AASHTO LRFD Table 4.6.2.2.1-1, case (j) is used. From Table 4.6.2.2.2b-1 the case with connection only enough to prevent relative vertical displacement at interface of girders is used:

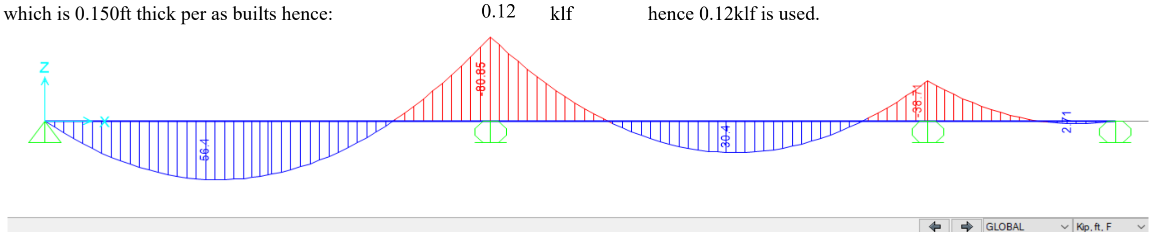
W	L	N_L	I	J	K	C	D	S	LLDF
ft	ft	#	ft ^4	ft ^4	#	#	ft	ft	#
61.25	77.75	3	4.7932	0.4467	3.59	2.83	9.29	5.8333	0.6277

includes DLA and LLDF

[G] CSi BRIDGE ANALYSIS RESULTS

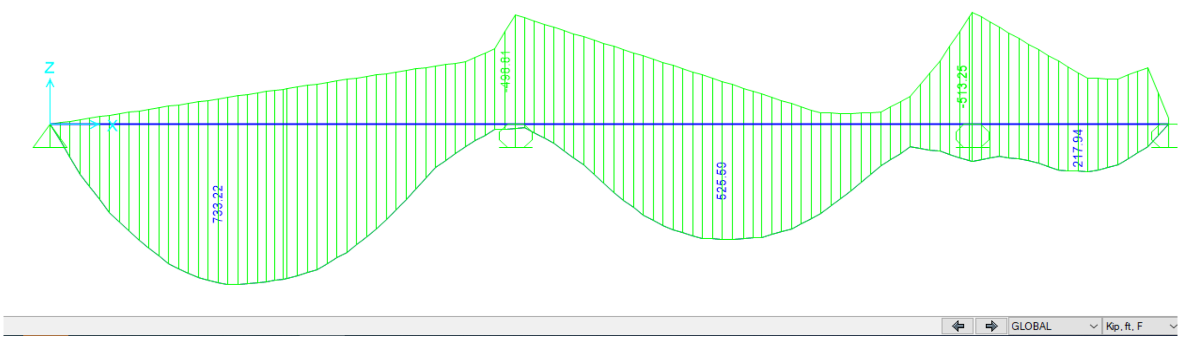
DW Moment diagram:

From structure cross section it can be seen that girders either receive utility loads, assumed to be 0.10klf per girder, or they receive paving load which is 0.150ft thick per as builts hence: 0.12 klf hence 0.12klf is used.



DW moment at pier 2: 80.85 k-ft

TYPE 3S2 (AASHTO 2) Moment diagram:

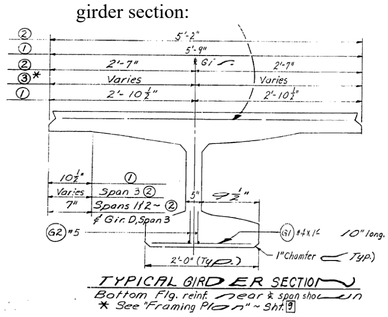


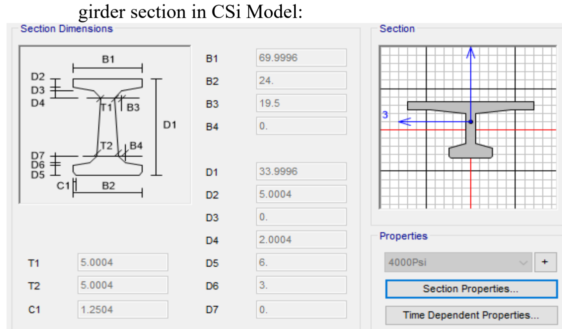
TYPE 3S2 moment at pier 2: 498.81 k-ft

Settlement Moment diagram:

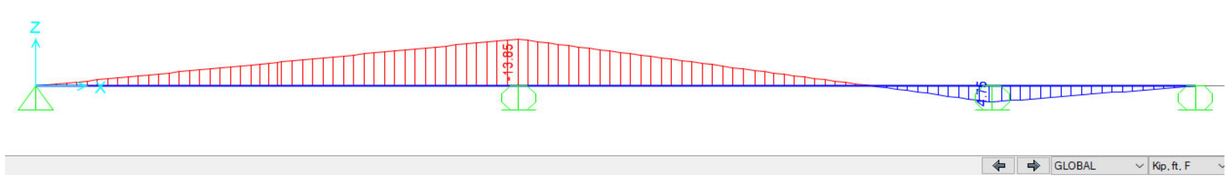
A settlement of 1/4" is applied to the abutment 1 and moment is determined:

girder section: girder section in CSi Model:





girder A =	675	sq.in
girder I3 =	99393	in^4
girder J =	9263	in^4



Settlemen (SE) moment at pier 2: 13.85 k-ft

[G] LOAD RATING CALCULATIONS

For Load and Resistance Factor method per BDM 13.1.1:

$$RF = \frac{C - \gamma_{DC}DC - \gamma_{DW}DW \mp \gamma_p P}{\gamma_{LL}LL(1+IM)}$$

Where:

- RF = Rating factor
- C = $\phi c\ \phi s\ \phi n\ R_n$, where $\phi c\ \phi s \geq 0.85$ for strength limit state
- C = f_R for service limit state
- R_n = Nominal Capacity of member
- f_R = Allowable Stress per LRFD
- DC = Dead load due to structural components and attachments
- DW = Dead load due to wearing surface and utilities
- P = Permanent loads other than dead loads
- LL = Live load effect
- IM = Dynamic load allowance (Impact)
- γ_{DC} = Dead load factor for structural components and attachments
- γ_{DW} = Dead load factor for wearing surface and utilities
- γ_P = Load factor for permanent load
- γ_{LL} = Live load factor
- ϕ_c = Condition factor
- ϕ_s = System factor
- ϕ_n = Resistance factor based on construction material

Condition Factor:

Condition Factor Per BDM = ϕc = 1.00 (Condition State 1 is stated in the inspection report)

System Factor:

System Factor Per BDM = ϕs = 1.00 (For superstructure- Flexure)

Dead and Live Load Factors:

	γ
Dead Load	1.25
Settlement	1.00
Superimposed Dead Load- DC (SDLC)	1.25
Superimposed Dead Load- DW (SDLW)	1.50

ADTT = 109 [INSP.]

Truck ADT = 7% [INSP.]

γ_{Legal} = 1.30 (γLL for Legal and NRL)

Nominal Capacity of Member: R_n = 583.19 k-ft

Rating Factor without settlement RF = 1.13 k-ft

Rating Factor with:	***0.25in***	***settlement***	RF = 1.10	k-ft
Rating Factor with:	***0.50in***	***settlement***	RF = 1.07	k-ft
Rating Factor with:	***0.75in***	***settlement***	RF = 1.03	k-ft
Rating Factor with:	***1.00in***	***settlement***	RF = 1.00	k-ft

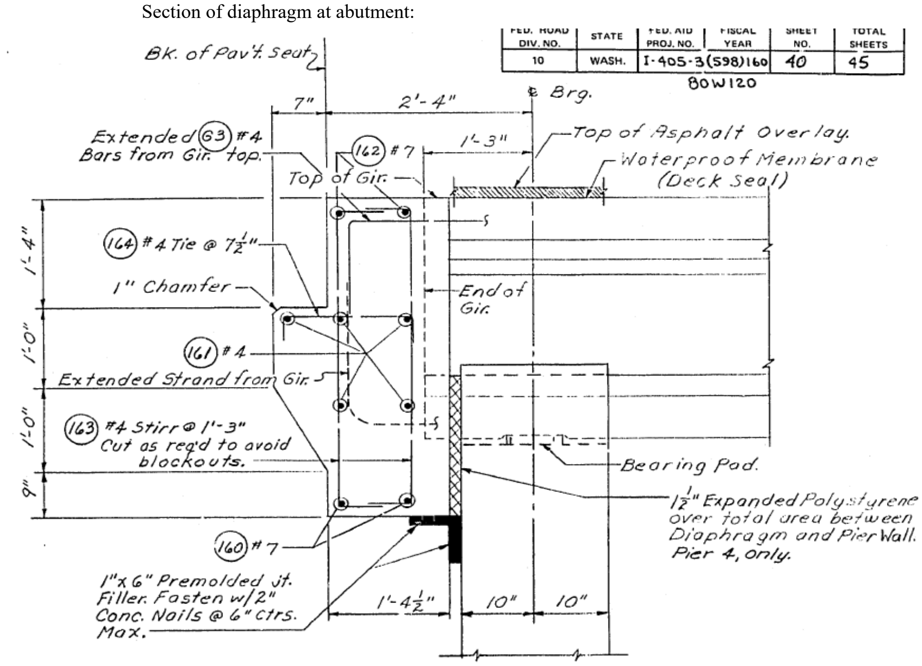
It is recommended that the Initial Threshold vertical movement for the Geotechnical Instrumentation Plan be 1/2" and that the Action Threshold (stop work) be 3/4". These values will ensure movements do not exceed RF = 1.0.

PARSONS		MADE BY	DATE	CHK BY	DATE
		N. Ala	12/02/2021	E. Kelley	12/03/2021
Job Number	WBS Number	TITLE	WSDOT - Renton to Bellevue		
650512	00531		Bridge 405-24 Monitoring Threshold		

[H] HORIZONTAL THRESHOLD FOR GIP

It is recommended that the Initial Threshold horizontal movement for the GIP be 1/4" and that the Action Threshold (stop work) be 3/8". These values were selected in consultation with the Geotechnical Engineer considering that the abutment footing lies behind the soil active failure plane. The abutment type (diaphragm beyond abutment stem) is suited to the anticipated movements meaning the expansion joint will not close and anticipated movements will not distress the bearing pads.

Section of diaphragm at abutment:



RETAINING WALL 06.50L

5.1 – Bridge 405-24 Girder M(-) Capacity

[A] BASIS

To determine the negative moment capacity of the girder mild steel that extends into the intermediate pier on BR 405-24.

[B] REFERENCES

Acronym	Source
AASHTO	AASHTO LRFD Bridge Design Specifications 8th Edition

[C] MATERIAL PROPERTIES

Concrete (Min. for Precast Girders)

$f'_{c,gird}$ = 7.0 ksi

Reinforcing Steel (Deck)

f_y = 60.0 ksi

ε_{cu} = 0.003 (concrete crushing strain limit)

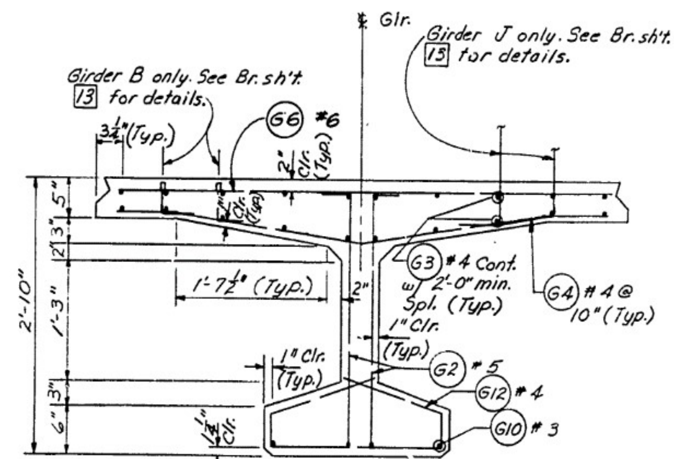
ε_{ttens} = 0.005 (tension controlled steel strain limit)

[D] GEOMETRY

Girder Depth, h =	34.00 in
Girder Top Flange Width, b_t =	62.00 in
Girder Web Thickness, b_w =	5.00 in
Girder Bottom Flange Width, b_b =	24.00 in
Uniform Girder Bottom Flange Thickness, t_{bf} =	6.00 in
Uniform Girder Top Flange Thickness, t_{tf} =	5.00 in

Concrete Cover

Top Cover, t_{ctop} =	2.00 in
Bottom Cover, t_{cbot} =	1.00 in



GIRDER SECTION

[E] EFFECTIVE FLANGE WIDTH [AASHTO 4.6.2.6.1]

	Exterior Girders
Half of Adjacent Girder Spacing, $0.5{*}S_{gadj}$ =	2.92 ft
Overhang Width, b_{OHmin} =	2.92 ft
Full Flange Width, b =	5.83 ft
Effective Flange Width, b^{ext}_{eff} =	5.83 ft

[F] TYPICAL REINFORCEMENT SUMMARY FOR NEGATIVE MOMENT REGIONS

Slab -Typical Topping Slab Reinforcement Summary

	n_{bar}	Bar Size	$d_{b,ts}$	$A_{b,ts}$	A_{s1}	
Layer 1 Top Flange Long =	10	#4	0.500 in	0.200 in²	2.000 in²	As1
Layer 2 Top Flange Long =	10	#4	0.500 in	0.200 in²	2.000 in²	As2

[G] ADDITIONAL LONGITUDINAL REINFORCEMENT FOR NEGATIVE MOMENT REGIONS

$\beta_1 = 0.85 - (f'_{c,gird} - 4)*0.05 =$ 0.70 [AASHTO 5.7.2.2]

$\phi =$ 0.90

d_1, Additional Top Longitudinal reinf. depth = 31.00 in (applies to A_{s1})

d_2, Additional Bottom Longitudinal reinf. depth = 29.00 in (applies to A_{s2})

$A_{s,tot}$	a_{rct} or $a_{t\text{-}beam}$	c	d_s	ε_t	Controlled?	M_n	ϕMn
4.0 in²	1.68 in N/A	2.40 in	30.00 in	0.0345	TENS.	583.2 k-ft	-524.9 k-ft

$a_{rct} =$ If($A_{s,tot}f_y/(0.85f'_{c,gird}b_{bf})>t_{bf}$,"NA",($A_sf_y/(0.85*f'_{c,gird},b_{bf})$)) [AASHTO 5.7.3.1.1-4]

$a_{t\text{-}beam} =$ If($a_{rct}<t_{bf}$,"NA",($A_{s,tot}f_y-0.85f'_{cgird}(b_{bf}-b_w)*t_{bf})/(0.85f'_{cgird}b_w)$) [AASHTO 5.7.3.1.1-3]

$c = \max(a_{rct}, a_{t\text{-}beam})/\beta_1$ [AASHTO-5.7.3.1.2-3]

$a = \beta_1 c$ [AASHTO-5.7.3.2.2]

$d_s = (A_{s1}d_1 + A_{s2}d_2)/(A_{s1}+A_{s2})$

$\varepsilon_t = (d_s - c)*\varepsilon_{cu} / c$ [AASHTO 5.7.2.1]

Tens. Controlled Section = $\varepsilon_{t} < \varepsilon_{ttens}$ [AASHTO 5.7.2.1]

$\phi = 0.75 \leq 0.75 + 0.15\,(\varepsilon_t - \varepsilon_{cl})/(\varepsilon_{tl} - \varepsilon_{cl}) \leq 0.9$ AASHTO C5.5.4.2.1-1]

$M_n =$ IF($a_{rct}>t_{bf}$,($A_{s,tot}f_y(d_s-a/2)+0.85f'_{c,gird}(b_{bf}-b_w)t_{bf}*(a/2-t_{bf}/2)$,$A_{s,tot}f_y(d_s-a/2)$) [AASHTO-5.7.3.2.3]

$\phi M_n = \phi*\max(M_{n,rct}, M_{n,t\text{-}beam})$ [AASHTO-5.7.3.2.2-1]